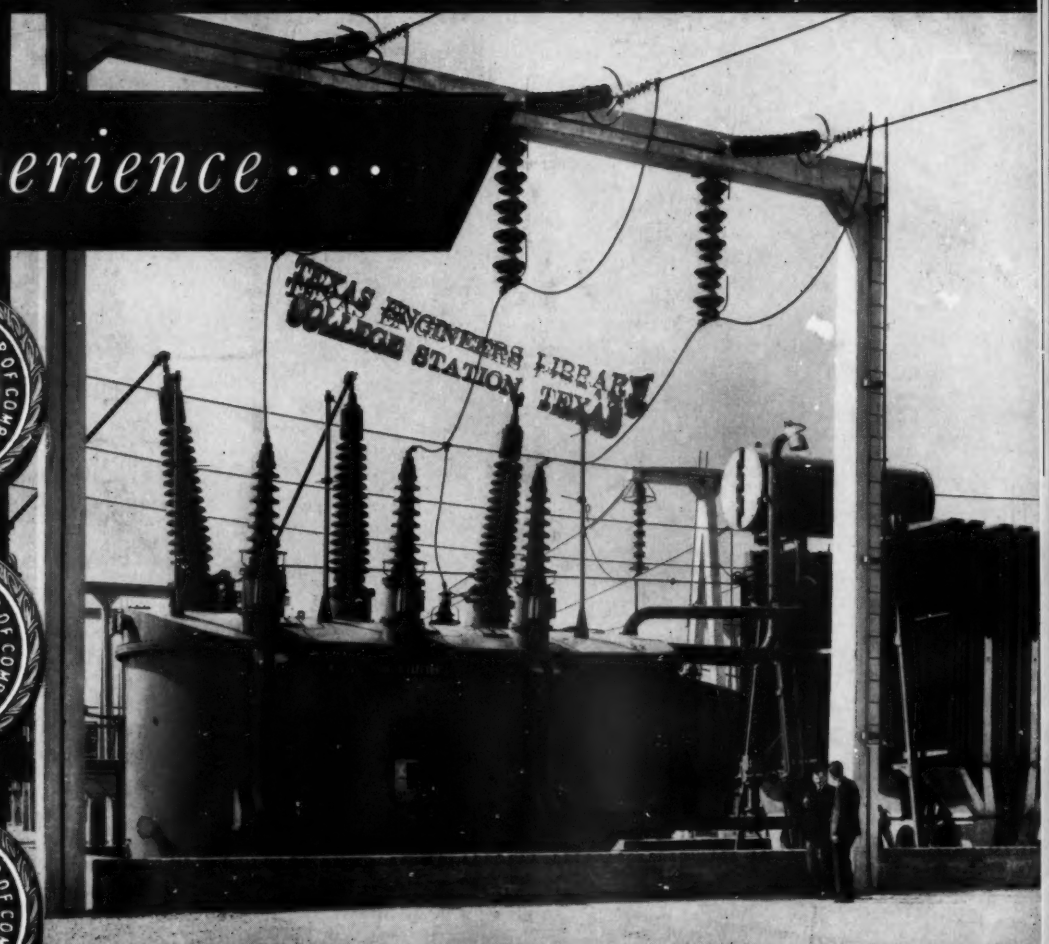
Experience . . .



were recently presented to men who design and build 'ENGLISH ELECTRIC' transformers. No less than 3750 man-years have been devoted by long-service employees in the advancement of

ENGLISH ELECTRIC'

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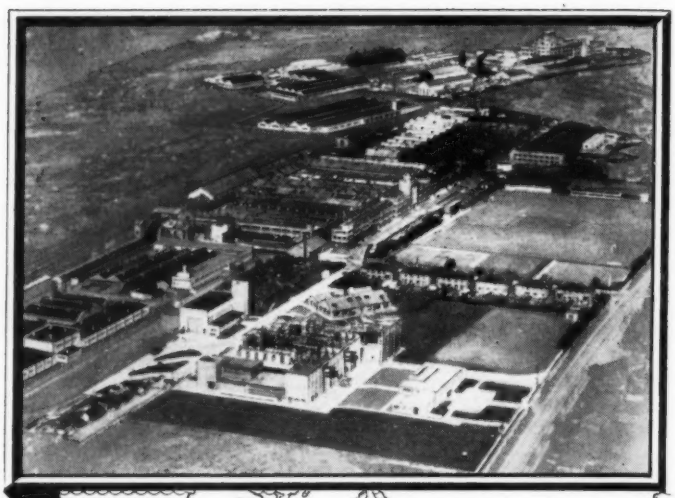
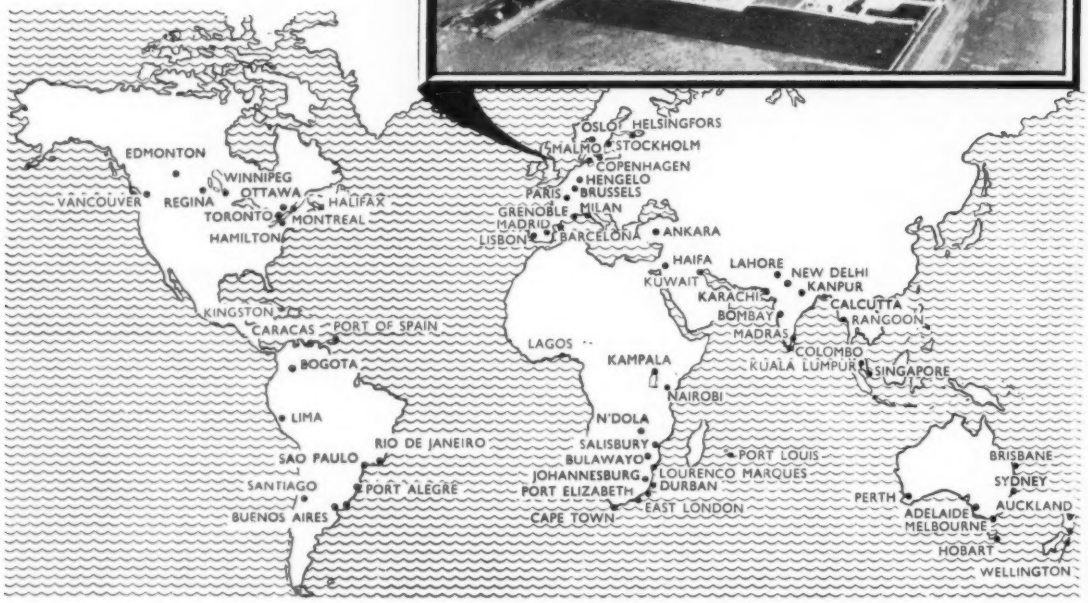
HEBBURN –

THE SOURCE

OF ELECTRICAL

SWITCHGEAR





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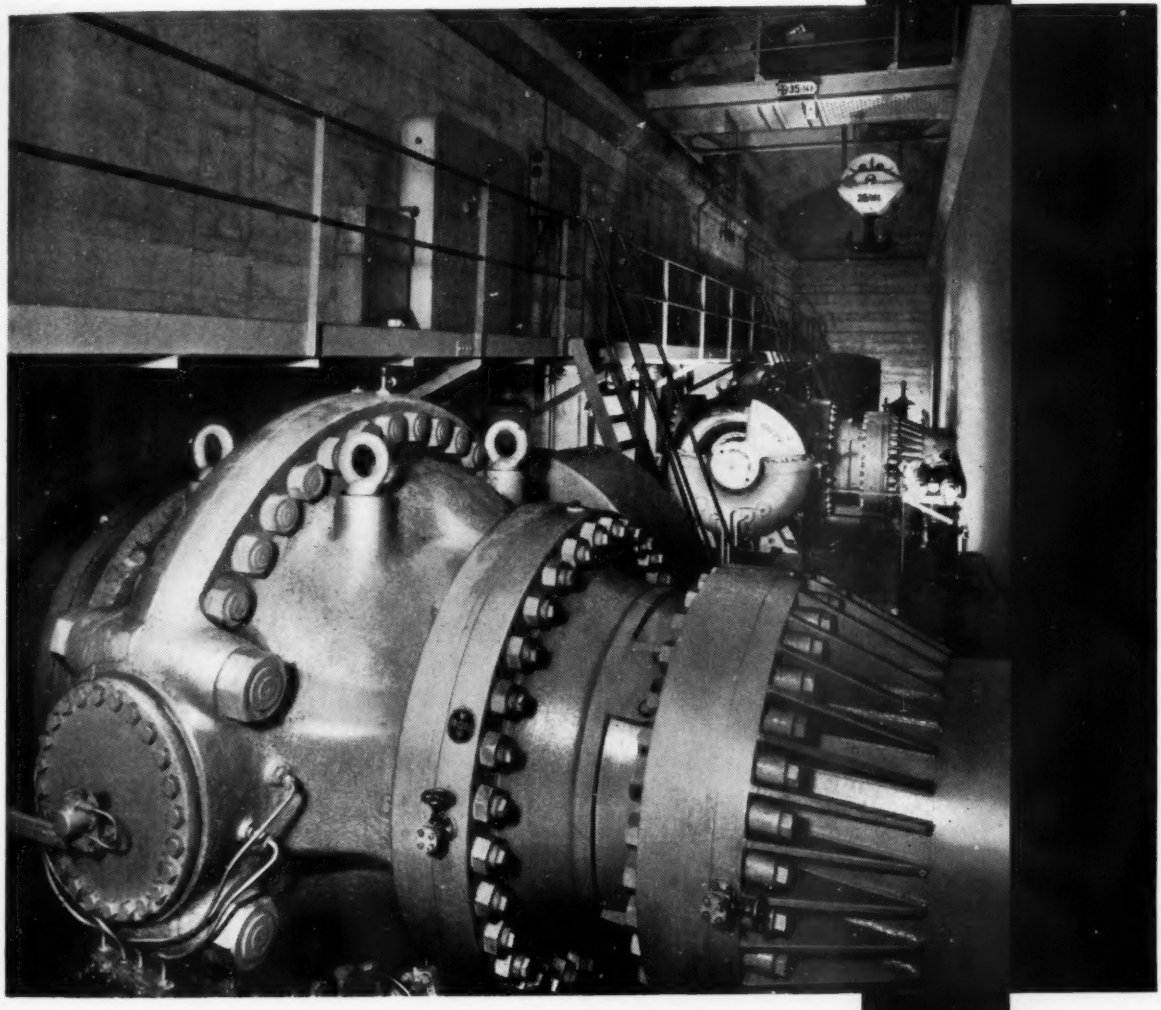
Equipment for Hydro-Electric Power Stations



Valve chamber with rotary slide sphériques valves as turbine shut-off device in a swiss power station

DE ROLL

Louis de Roll, Ironworks Ltd.,
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Complete barrage installations; gates and hoisting equipment, gantry- and overhead travelling cranes, trash screen cleaning machinery; sluice gates; control valves for high pressure conduits; automatic safety organs for pipe lines; discharge regulating valves, rotary valves with rotary-piston servomotors, needle valves, butterfly valves, high pressure fittings, piping systems for oil and cooling water plant; funicular and aerial cableways for passenger and goods transport; building machinery for dam-building work.

76

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PRICE at a new competitive level

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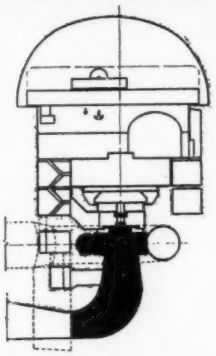
KARIBA—ON SITE—ON SCHEDULE

The Stay Ring for No. 1 Turbine at Kariba—shipped from England in May, 1958—is now in position. It is shown here installed in the vast underground power station, ready for the next major step, fabrication of the spiral casing.

This is the first of six Boving Turbines being installed at Kariba. Each is of 140,000 b.h.p.



SECTION THROUGH UNDERGROUND POWER HOUSE

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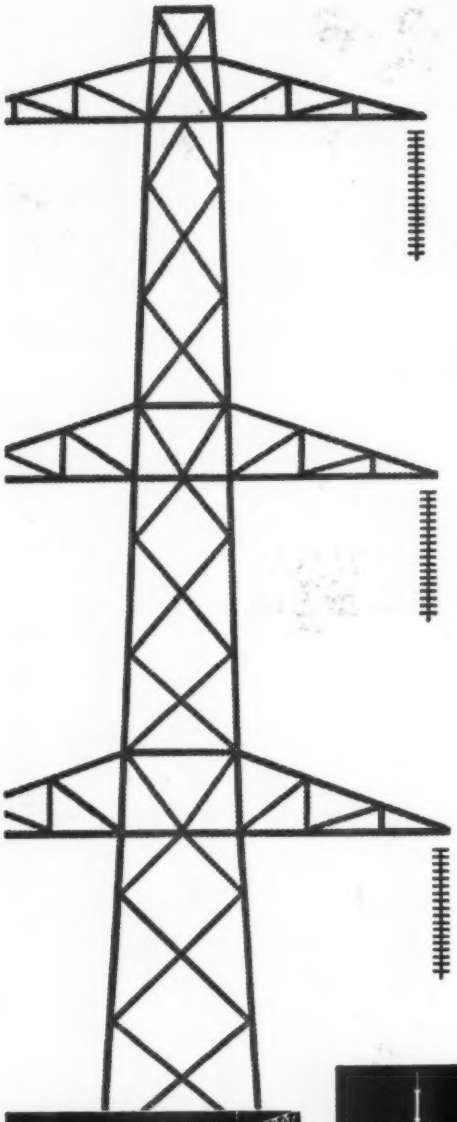

structural steelwork

DESIGN
FABRICATION
GALVANISING
INSTALLATION

by the **BICC** GROUP

Much of Britain's Grid and Supergrid systems and thousands of miles of overhead transmission lines overseas have been installed by the BICC Group which embraces

BRITISH INSULATED CALLENDER'S CONSTRUCTION CO. LTD.
and
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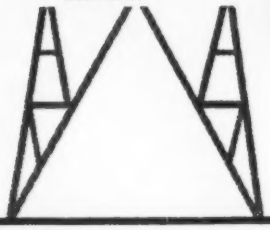
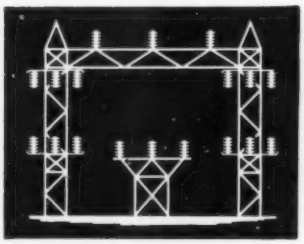
These two important members undertake between them the design, supply and erection of galvanised structural steelwork anywhere in the World.

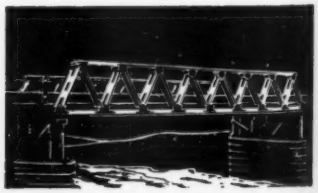
GANTRIES



MASTS & TOWERS



SUBSTATION STRUCTURES



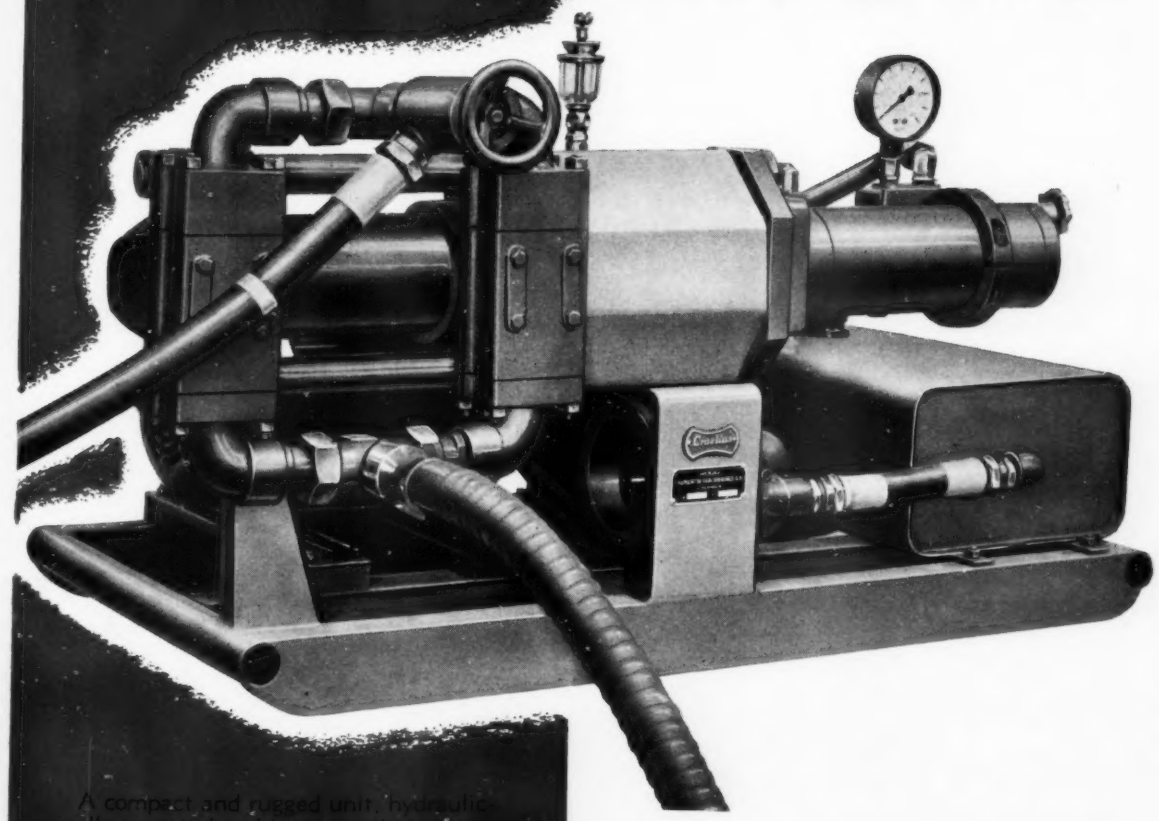
CALLENDER-HAMILTON BRIDGES

Contractors : BRITISH INSULATED CALLENDER'S CONSTRUCTION COMPANY LIMITED
30 LEICESTER SQUARE, LONDON W.C.2. TELEPHONE: TRAFALGAR 7777 CABLEGRAMS: BICALCON, LONDON.
Steelwork : PAINTER BROTHERS LIMITED, HEREFORD. TELEPHONE : HEREFOKD 3101

Automatic pressure control

Craelius

ZH-50 Grouting Pump



A compact and rugged unit, hydraulically operated and incorporating a completely dependable system of automatic pressure control whereby:

(a) The grouting pressure can be varied and automatically controlled during operation.

(b) The grouting pressure can be limited to any predetermined value in the operating range.

All risks of structural failures due to harmful surges, hydraulic shock or sudden pressure build-up eliminated.

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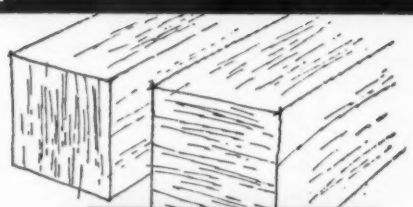
THIS is the compressor that costs less to maintain: the "Power Vane" Rotary. The essential simplicity of the rotary principle, pioneered by Consolidated Pneumatic, means that there are not only fewer parts to service (no valves, pistons, clutch or crankshaft) but also fewer replacement parts to stock. Oil-flood lubrication further reduces wear to a minimum, making "Power Vane" Rotary compressors good for long, continuous running without major attention. "Power Vane" Rotary compressors deliver pulsation-free, *cool* air in volumes from 125 to 600 c.f.m. F.A.D. For full details of all models, ask for Catalogue No. 58.

POWER VANE
ROTARY COMPRESSORS

Cost less to maintain



MANUFACTURED BY Consolidated Pneumatic

CONSOLIDATED PNEUMATIC TOOL CO LTD · DAWES ROAD · LONDON · SW6

Selecting Demolition and Construction Equipment

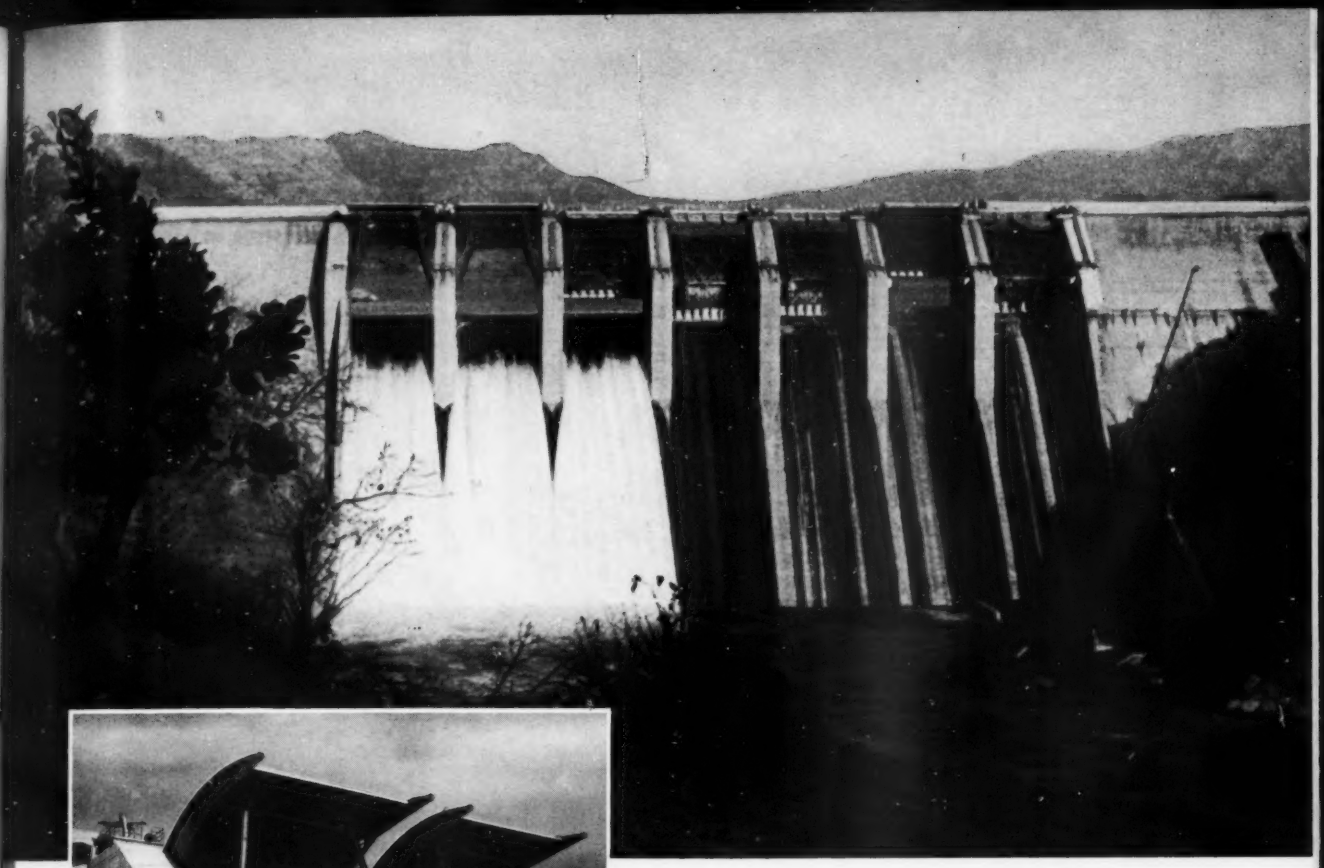
The full range of CP equipment for the contractor is too extensive for more than a passing reminder in this column. In citing one or two examples we invite you to ask for literature detailing the full range of rock drills, picks, pumps, vibrators, claydiggers, concrete breakers, sheeting drivers, woodborers and the like.

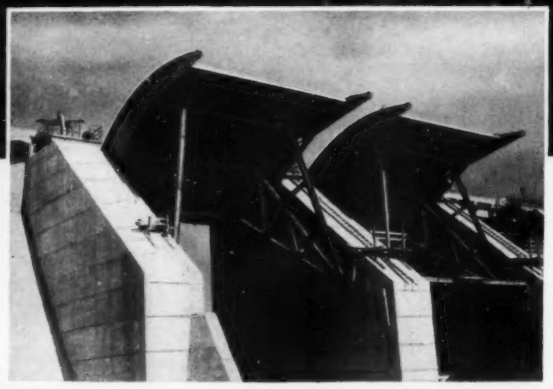
A Choice for Claydigging

Every contractor will be aware of the big variations in clay and other soft grounds and will thus appreciate the need for more than one type of claydigger. In this range of CP tools there is a model to suit every condition. The FL-22 Claydigger, for example, is ideally suited to clay tunnelling. CP representatives will be pleased to advise on the correct choice, or literature will be sent on request.

Choose Pneumatic Pumps for Constant Duty

Of all the many forms of pumps available for general pumping duties, the CP range of sludge and sump pumps ranks high for sheer reliability in service. The No. 77 sludge pump will operate 24 hours a day under the roughest conditions dealing with water containing up to 15% solids. The Nos. 1, 2 and 41 Sump Pumps are equally reliable for high or low lift operation. Write for literature giving the full range of capacities.





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By-pass gates, safety gates
Valley storage dams
High-pressure piping (penstocks)
Dock gates
Ship lifts
Slipways
Floating cranes
All hydraulic structures (including mechanical and electrical equipment) are supplied and erected ready for operation.

Photos 1 and 2
7 spillway tainter gates 12 x 12 m for dam flood control

Photo 3
Wye pipe for hydraulic power conduit
Inlet opening 7010 mm ø
Outlet opening 4240, pressure 7.6 atm. g
Rate of discharge 17 cu. m/sec.

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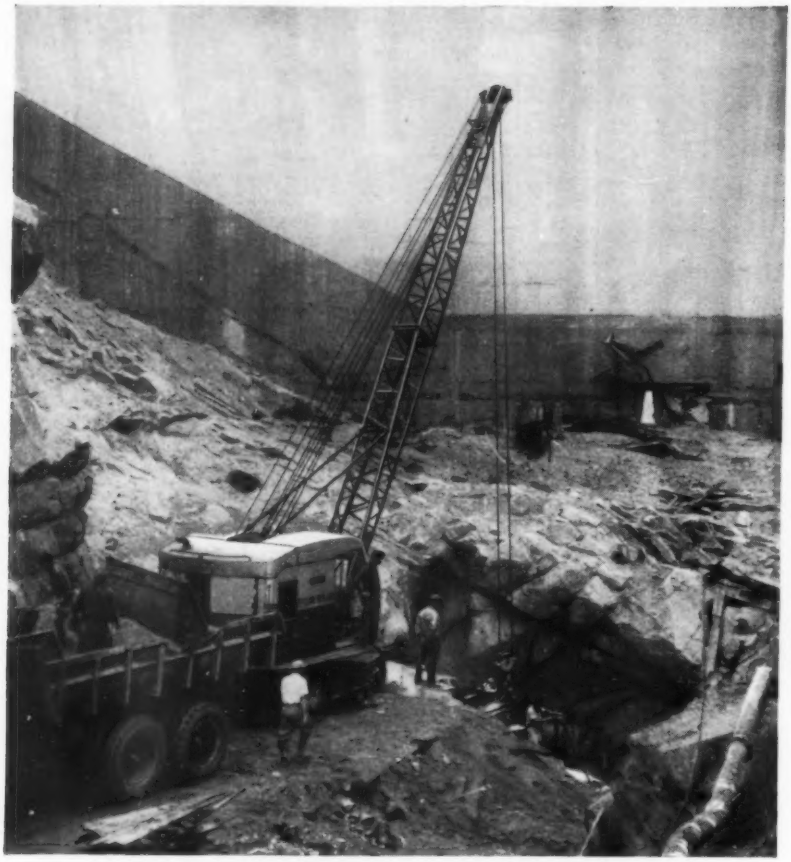
Ruston-Bucyrus

MOBILE LIFTING CRANES



19-RB operating as a skip crane at the foot of the Chicamba Dam in Portuguese East Africa. This concrete dam spans the River Revue near the Rhodesian border. The rock is broken up by compressed air drills and hand loaded by native labour into Euclid 9 yd. trucks.

22-RB lifting crane on the extension to a Hydro-electric scheme at Dalwhinnie, Scotland.

3 TO 40 TONS CAPACITY

for plant hire, steel erection, skip and bucket handling, placing pre-cast concrete sections as applied to Building and Public Works, Bridges and Dams, Power Stations, Oil Refineries

Among the series of excavator-type cranes offered by Ruston-Bucyrus there will probably be one to suit your purpose. In most sizes there is the choice of standard and oversize caterpillar mountings. The 22-RB is available as heavy-duty crane on caterpillars or truck-mounted. Send for publication RB1604 which gives introductory information on the complete range.

RUSTON-BUCYRUS

Ruston-Bucyrus Limited

EXCAVATOR SPECIALISTS, LINCOLN

R85A

64'000 HP Pelton Turbine Göschenen

TRADITIONAL SWISS ENGINEERING SKILL AND QUALITY

A century of experience in design and construction of waterpower-equipment and constant research in two hydraulic laboratories, one of them especially equipped for cavitation tests, assure the reliablity and efficiency for which Bell turbines are renowned

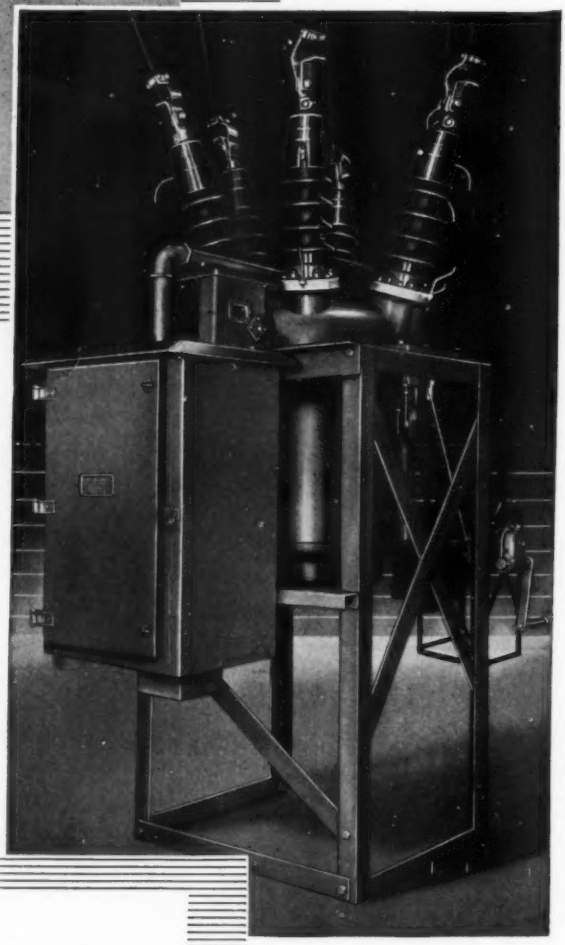
KRIENS Bell

Bell builds:
Water Turbines, Spherical straight flow valves, Butterfly valves, Penstocks and supply pipings, Pressure tunnel liners, Partially or fully automatic and remote controlled plants, Power house cranes, Towers for transmission lines, Water control equipment

Some projects with Bell Equipment:
Francis Turbines - Guyabo (El Salvador) head 56 m, 24'800 HP - Bärenburg (Switzerland) head 340 m, 85'000 HP (Supply together with Charmilles)
Pelton Turbines - Göschenen (Switzerland) head 698 m, 64'000 HP - Sedrun (Switzerland) head 588 m, 70'000 HP
Kaplan Turbines - Wildegg Brugg (Switzerland) head 15,3 m 34'000 HP - Petäjäskoski (Finnland) head 19 m, 67'600 HP

Bell Engineering Works Ltd.
Kriens/Lucerne (Switzerland)

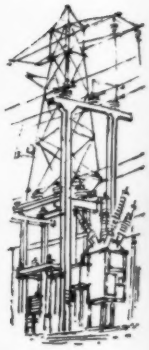
Why TYPE ROP 32 OUTDOOR SWITCHGEAR ?



When a decision must be reached concerning the installation of new, or replacement, OUTDOOR Switchgear the question may well be asked, "Why choose Type ROP.32?" For the very good reason that this class of Switchgear has acquired for itself a high reputation by giving reliable service in many parts of the World, and in varied climatic conditions.

Alternative Operating Mechanisms

The type ROP 32 circuit-breaker can be supplied with (a) Solenoid, (b). Hand wound or motor wound, or (c) Pneumatic closing mechanisms.



FOR 22/33 kV SERVICE

AEI

Manufactured at the Higher Openshaw Works of

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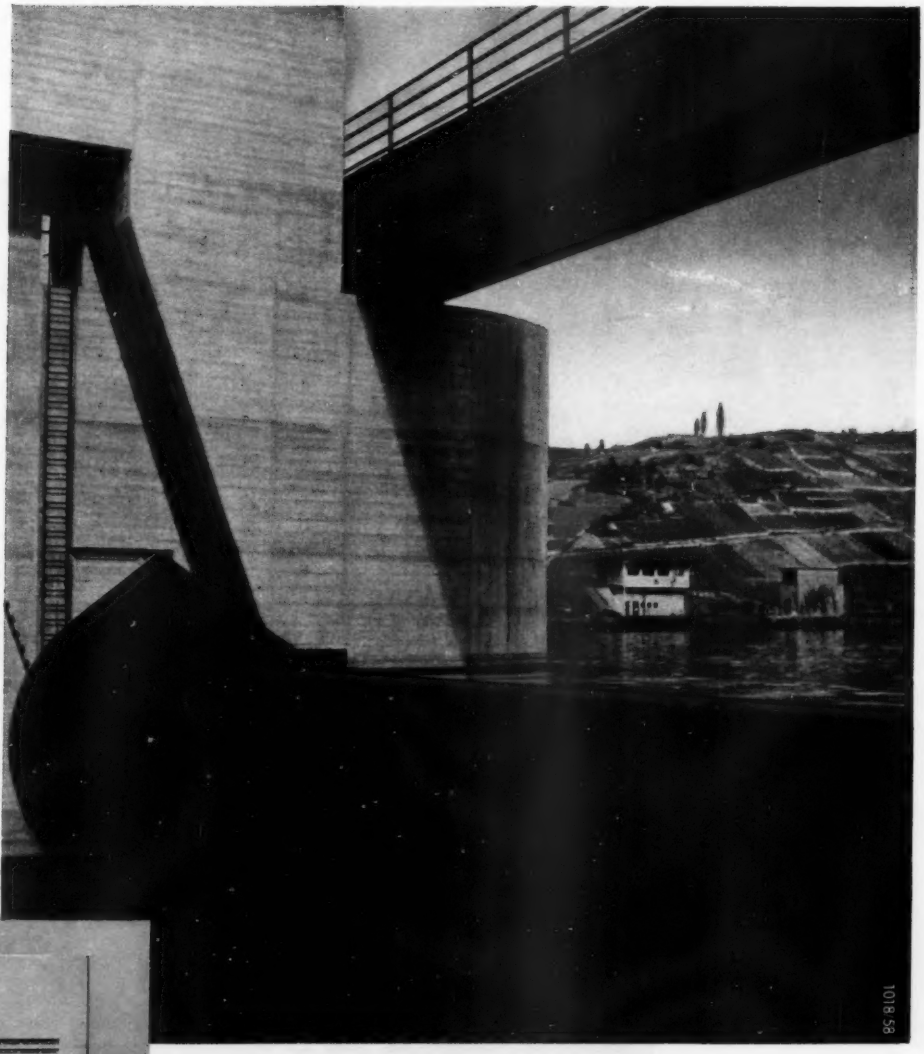
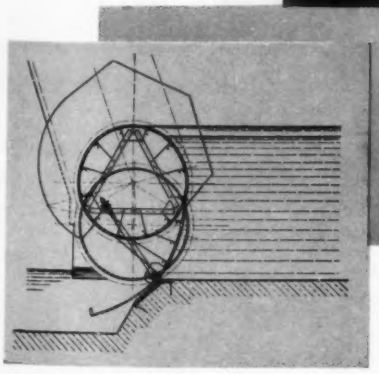
SWITCHGEAR DIVISION

Higher Openshaw, Manchester · Trafford Park, Manchester · Willesden, London

M·A·N





HYDRAULIC STRUCTURES

Roller gates particularly lend themselves for installation in rivers where sediment drift and ice prevail. For fine regulation of the water level gates can either be designed as submersible rollers or fitted with superposed flap.

Illustration shows roller weir in River Main near Randersacker, consisting of:

1 roller gate of 25.0 m clear width and 5.15 m water head, (gate can be lowered by 1.3 m),

1 standard type roller gate of 25.0 m clear width and 5.15 m water head,

1 standard type roller gate of 30.0 m clear width and 5.15 m water head.

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Better see us about the castings. Our design staff understand both sides of the problem. They know every aspect of foundry practice and they're sound engineers in their own right too. They can translate your exact specification into a cast form entirely free from structural fault. The rest is up to our metallurgists and foundry engineers. Their aim is quality and accuracy and they hit the mark every time. Birkett, Billington and Newton's technical teamwork can tackle *all* your problems—and solve them.

CASTINGS FROM A FEW OUNCES TO 10 TONS . . .

in phosphor-bronze, gun-metal, aluminium-bronze, manganese-bronze and light alloys. Precision machined bushes and bearings. Specialists in high-tensile aluminium-bronze castings, centrifugal-cast wheel blanks, shell moulded castings and chill-cast rods and tubes. Continuous cast phosphor-bronze bars up to 12 ft. lengths.

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SOCIETA' IDROELETTRICA MEDIO ADIGE, VERONA (ITALY)

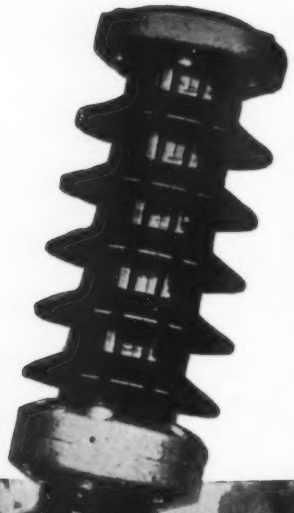
POWER STATION OF ISOLA SERAFINI
ON THE PO RIVER

KAPLAN TURBINES, 12,500 kW. each

Delivery: 230,000 lt/sec.
Head: 6·00 m.
Speed: 53·57 r.p.m.

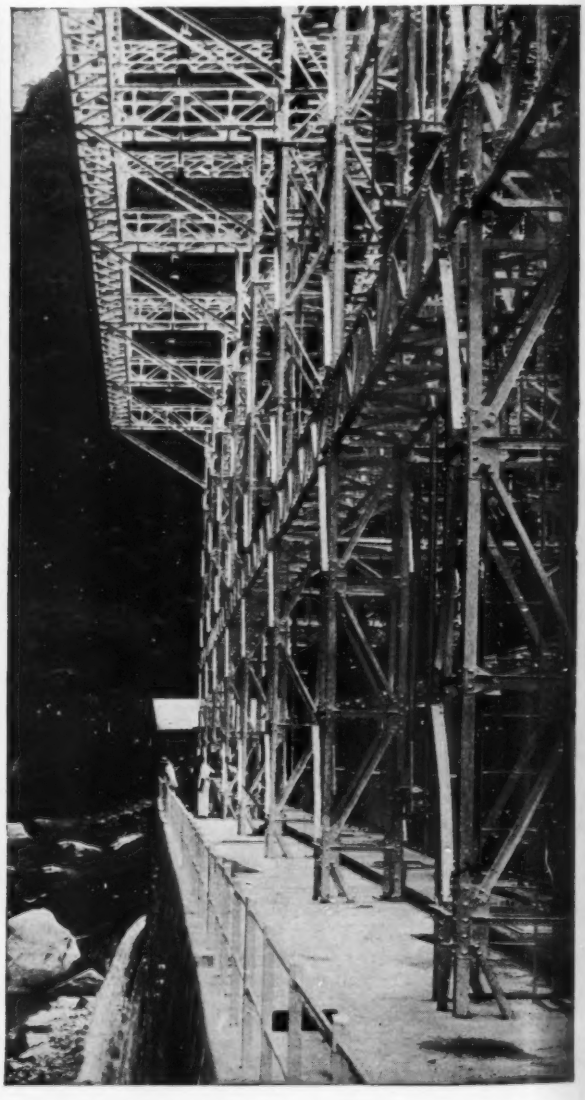
4 blades undergoing roughing-out process. Outside diameter of the wheel: 7,800 mm.

FRANCO TOSI S.p.a ★ LEGNANO (ITALY)



S.P.P. Post Insulators were used on all the 110kv outdoor switchgear on the Jog Falls Hydro-electric Scheme

The outdoor switchgear at Jog Falls Power Station, in Mysore State, India, equipped with S.P.P. Cylindrical Post Insulators type P.701.



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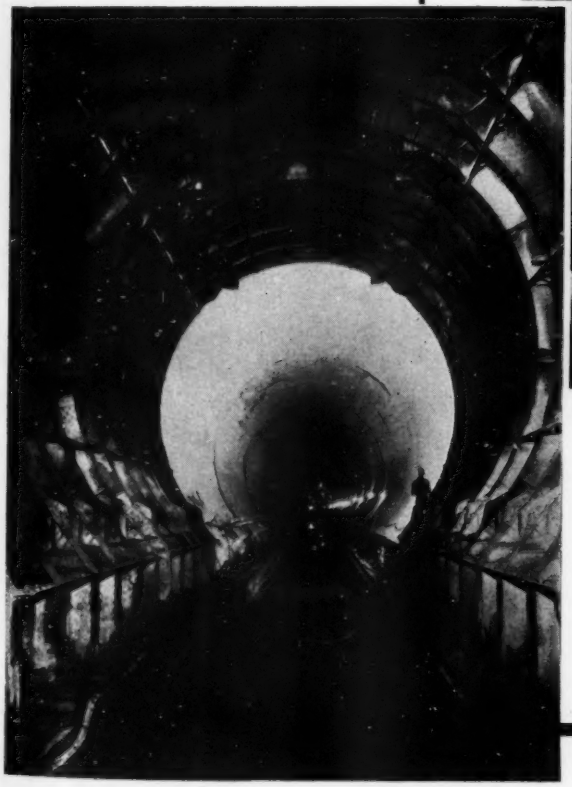
Laksapana Power Plant (Ceylon): Penstocks

Serre-Ponçon Dam Project : concrete gallery extending over 7,000 ft.

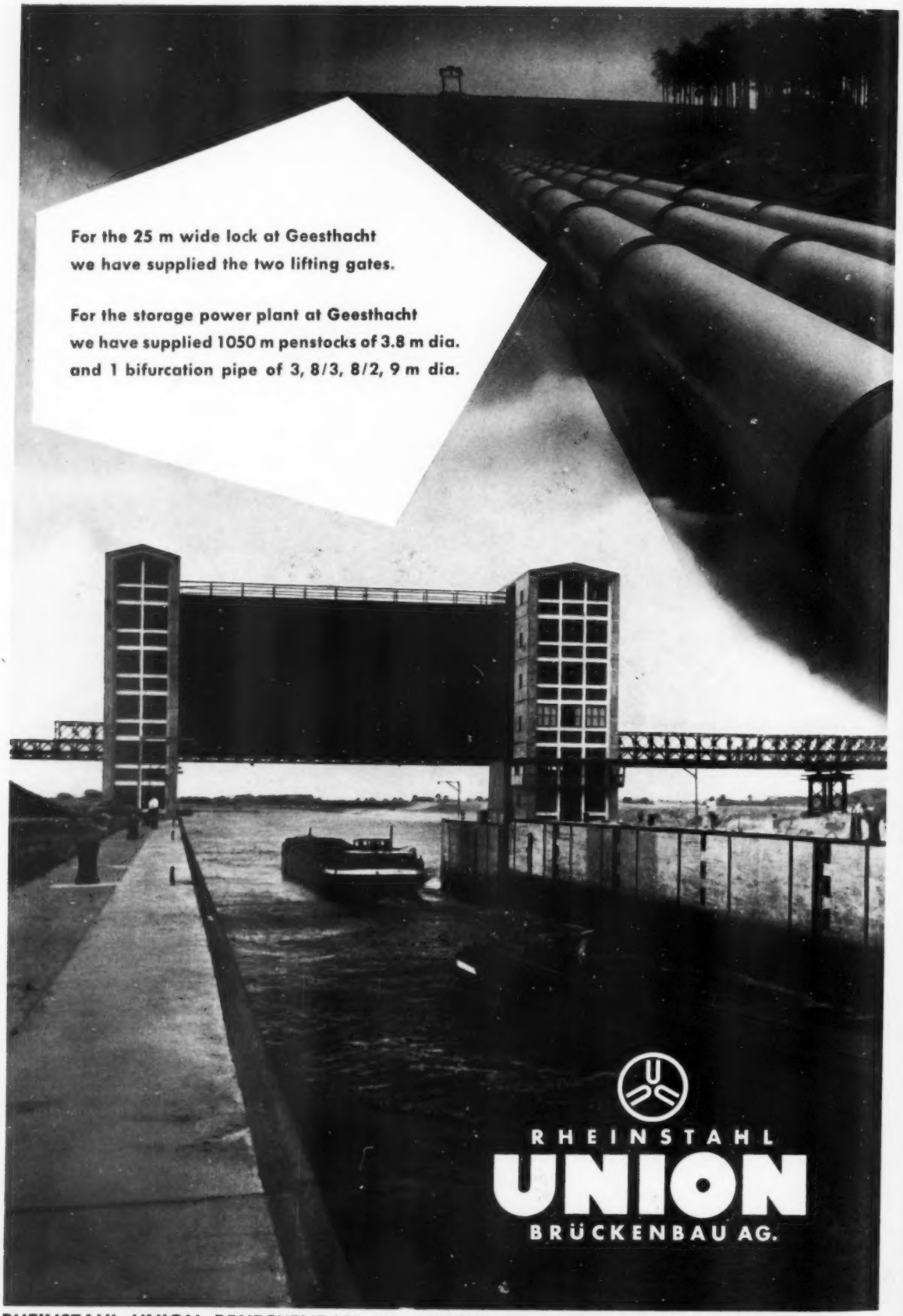


Laksapana Power Plant : Two 16,100 kW Pelton units under erection

CONSTRUCTION AND ERECTION OF ANY HYDRAULIC POWER PLANT EQUIPMENT

Penstocks, Valves, Turbines, Alternators, Electric Switchgear, Transformers, etc...

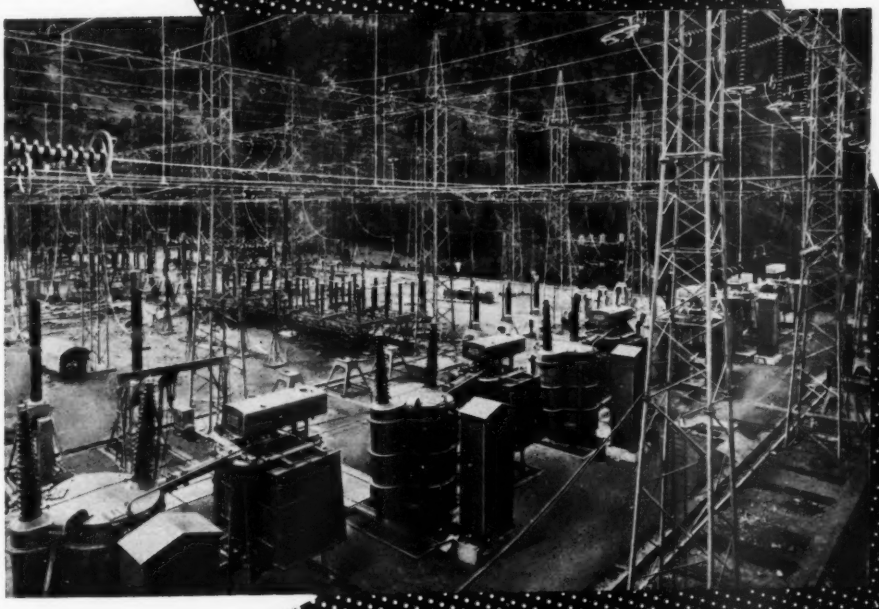
CIVIL ENGINEERING WORK


For the 25 m wide lock at Geesthacht
we have supplied the two lifting gates.
For the storage power plant at Geesthacht
we have supplied 1050 m penstocks of 3.8 m dia.
and 1 bifurcation pipe of 3, 8/3, 8/2, 9 m dia.
RHEINSTAHL
UNION
BRÜCKENBAU AG.

RHEINSTAHL UNION BRUECKENBAU AG., WORKS AT DORTMUND AND GELSENKIRCHEN

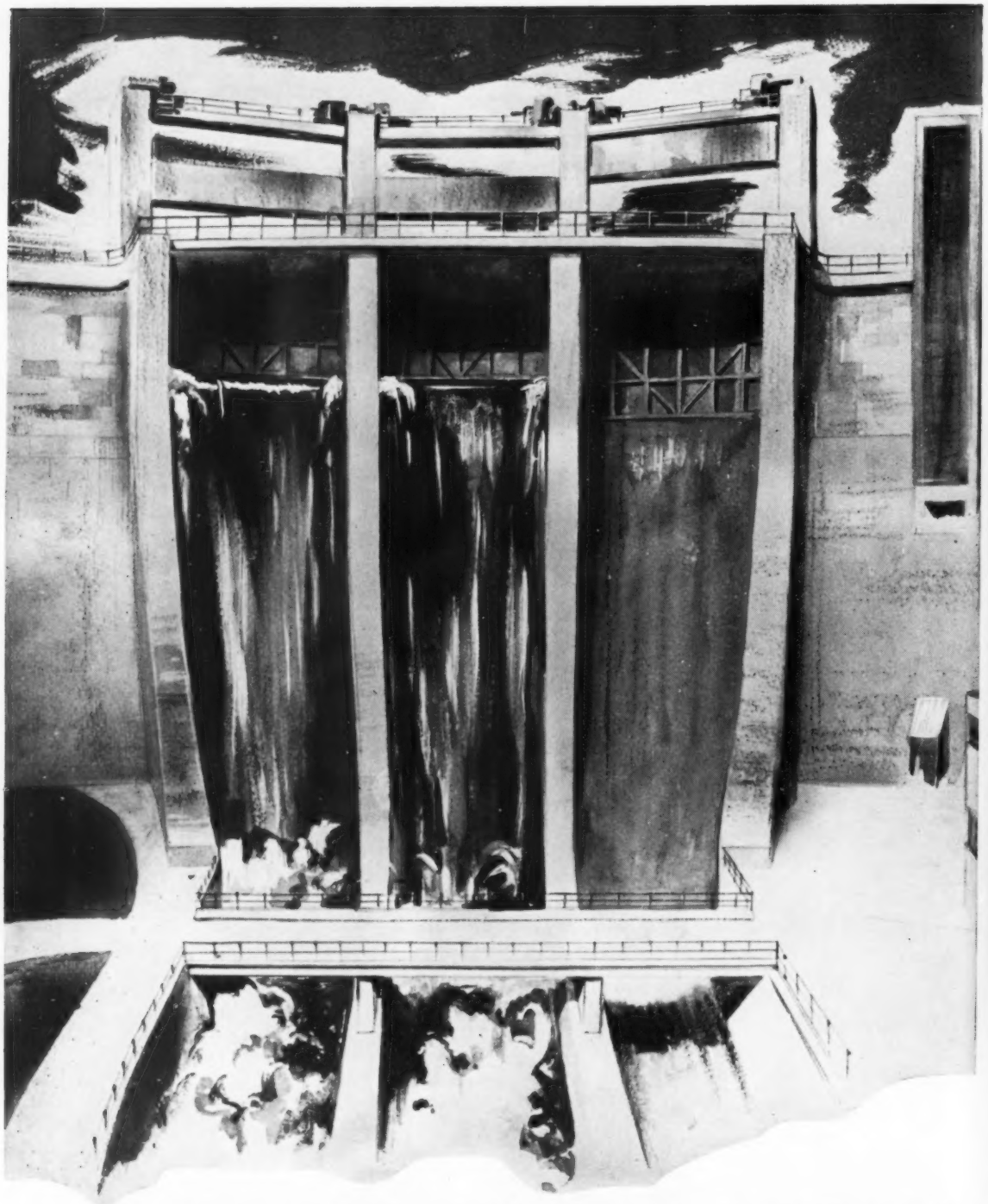
savigliano

260 kV class
transformers



società
nazionale
delle officine
di
savigliano
corso mortara 4
torino (italy)

electromechanical constructions



TO DOUBLE THE POWER OF ROXBURGH

AEI

Four AEI waterwheel alternators at the New Zealand Electricity Department's Roxburgh Power Station are already feeding 160,000 kW into the South Island grid. Four additional sets, to bring the station up to its designed capacity of 320,000 kW, are now in course of construction.

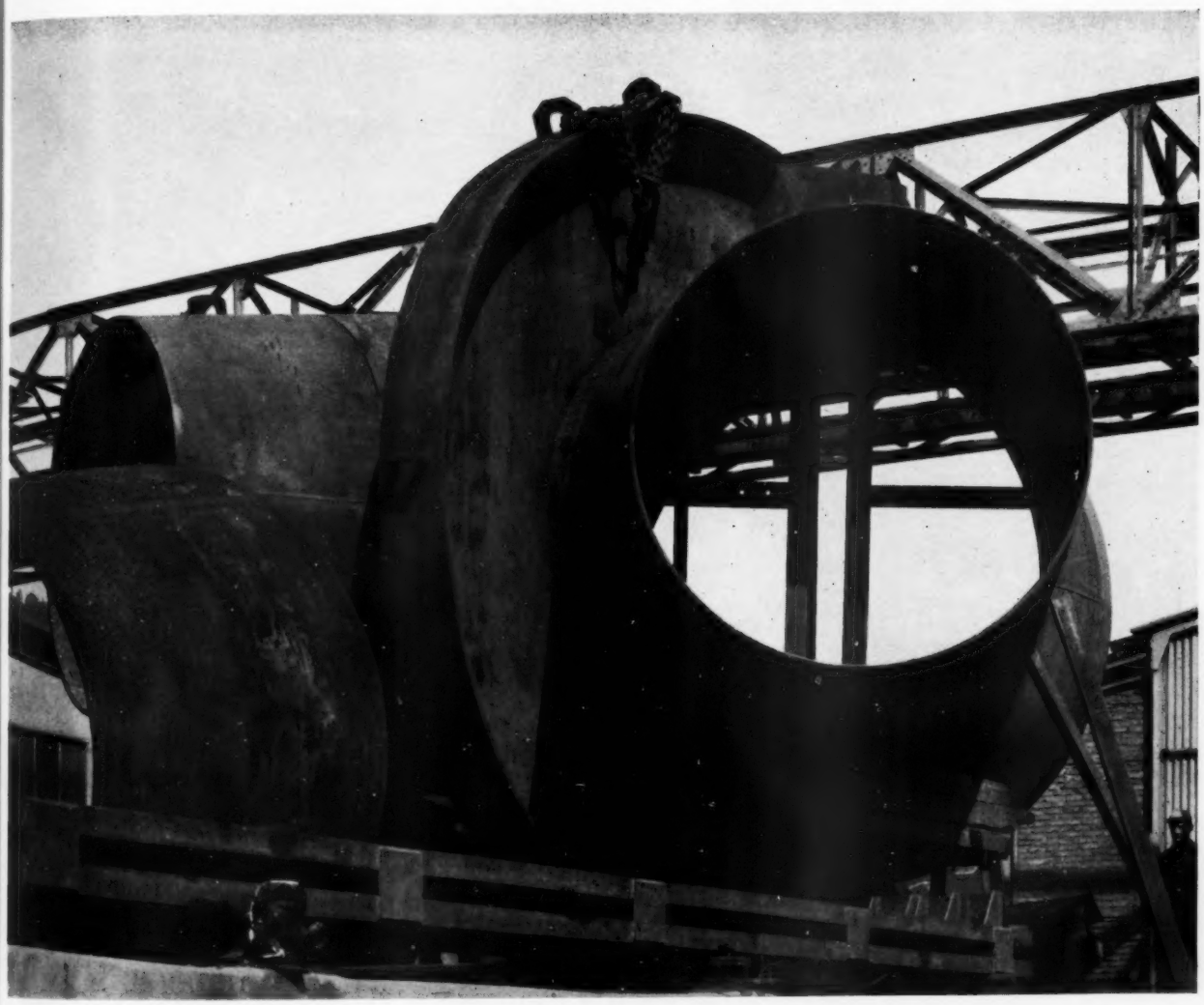
ASSOCIATED ELECTRICAL INDUSTRIES LIMITED

HEAVY PLANT DIVISION — RUGBY AND MANCHESTER, ENGLAND

A 5379

Y-PIPE FOR PRESSURE PIPELINE

to Power Station in Jugoslavia



DESIGN AND CONSTRUCTION:

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AKTIENGESELLSCHAFT

WIEN **GRAZ**

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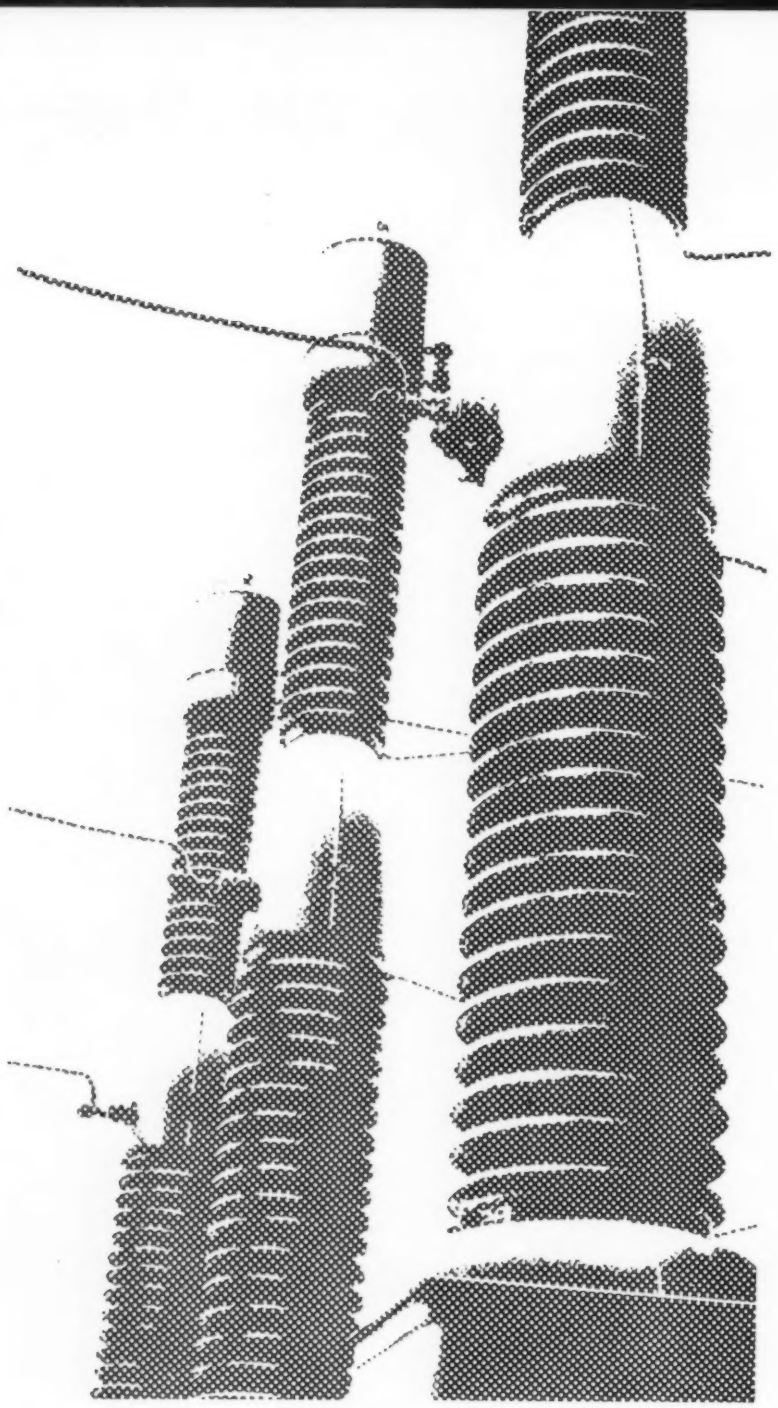

ELECTRICAL EQUIPMENT

CIRCUIT BREAKERS,
INSTRUMENT TRANSFORMERS,
ISOLATING SWITCHES,
BUSHING INSULATORS,
CONTROL DESKS AND PANELS,
SWITCHBOARDS,
FOR MEDIUM, HIGH AND
ULTRA-HIGH VOLTAGES

OFFICINE ELETTRO MECCANICHE GALILEO
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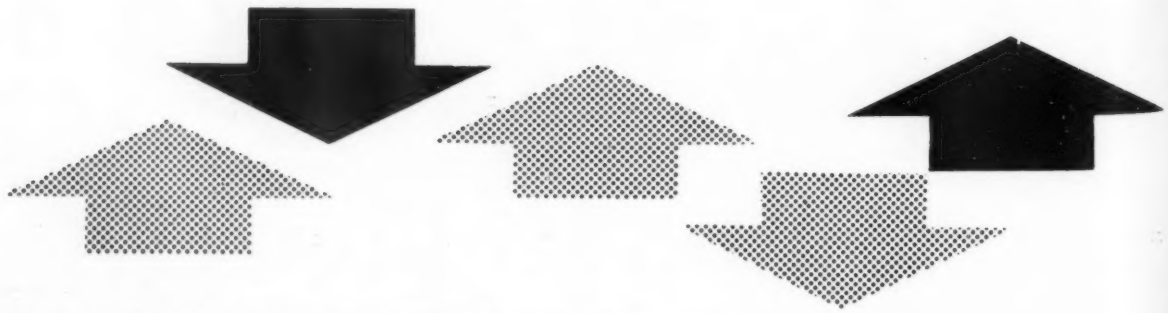

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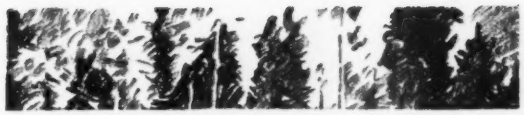
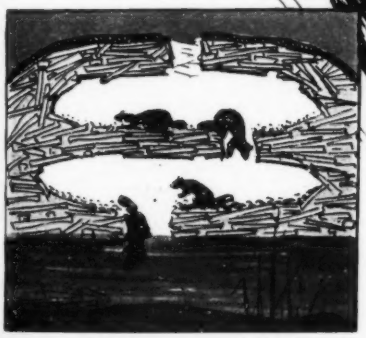


ACHIEVEMENT IN THE WILDERNESS



Cemented with frozen mud, the beaver's lodge shelters him from the elements and his many enemies. The lower chamber, entered from underwater, is his 'dining room,' while above it, the sleeping space is lined with cedarwood shavings or peeled twigs.

GIVE the beaver a valley, a stream and enough trees and there's no limit to the efforts he'll make. Dams a thousand feet long containing as many tons of material, holding back 'ponds' thousands of acres in extent—canals, stretching for hundreds of yards to a particular stand of timber—two-storey 'houses' sometimes forty feet in diameter—a strange and wonderful 'civilisation' flourishing where once the forest stood unchallenged.

On an infinitely greater scale but with the same inexhaustible energy and aptitude for the job we are creating the mechanism of modern life in places which for centuries have withstood the encroachment of man. Like the beaver, we are eager to build —and the wilderness is a challenge to be met and conquered.

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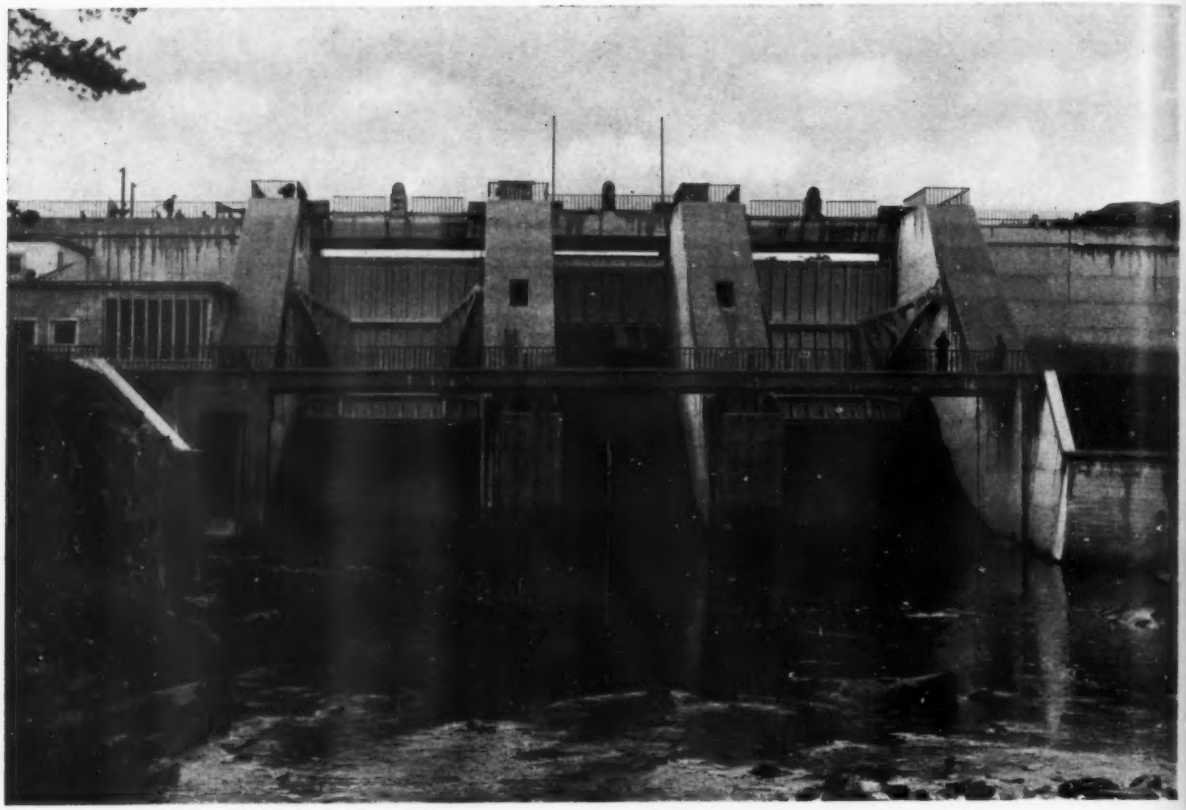

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are installed in many important hydro-electric works, at home and abroad, to give efficient and reliable control of spillway and culvert discharge. Types supplied include sliding, fixed-roller and free-roller Direct Lift; Drum; Radial and Tilting.

Illustrated above are the Dundreggan Works of the Moriston Project in Scotland, that incorporate two 28 ft. 6 in. span × 27 ft. deep radial gates and a 20 ft. span × 10 ft. deep tilting gate. The other photograph shows one of six 9 ft. 10¼ in. span × 16 ft. 10¾ in. deep free-roller gates for low level flood release, in operation on the Owen Falls Scheme in Uganda.

Glenfield

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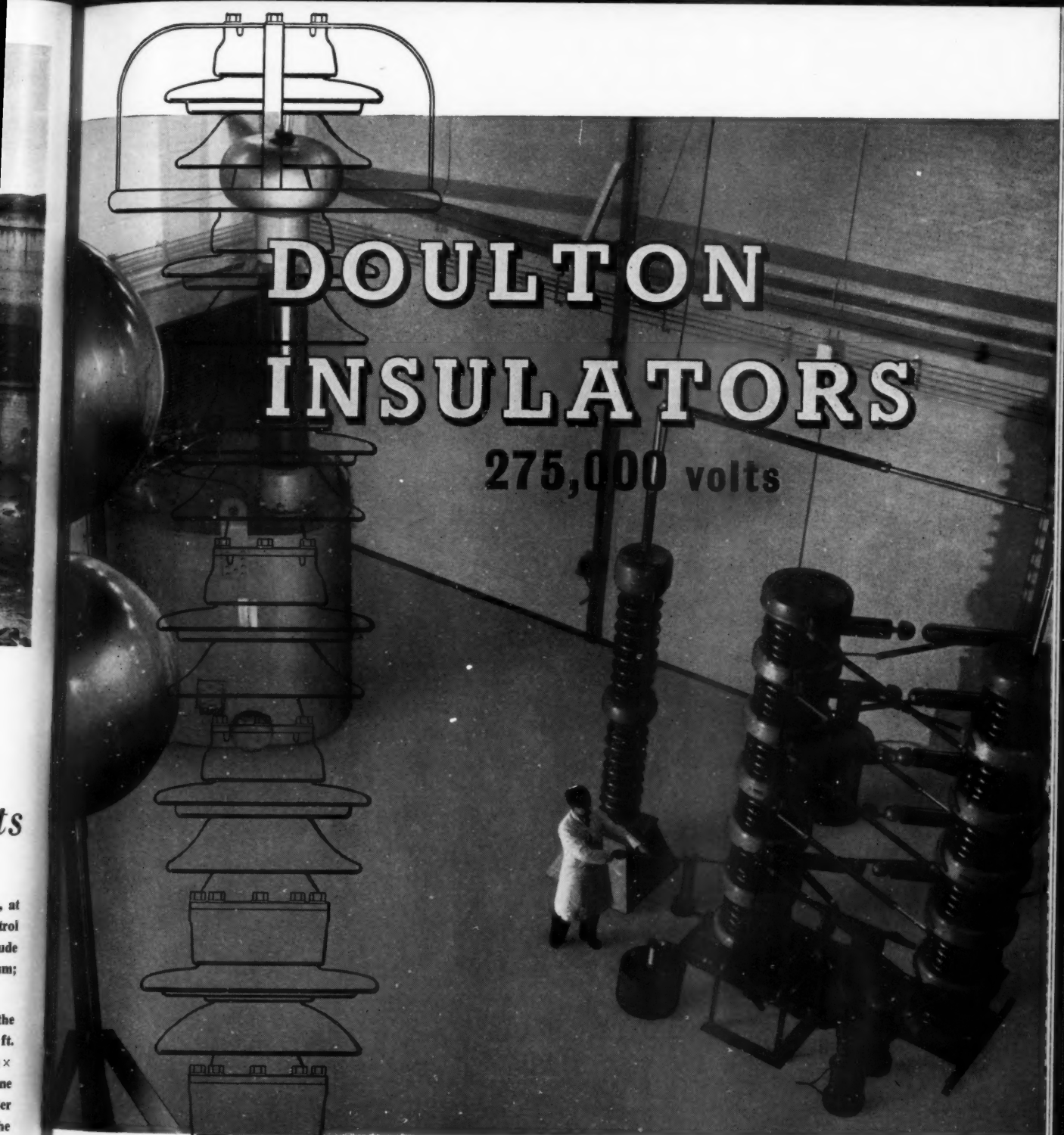

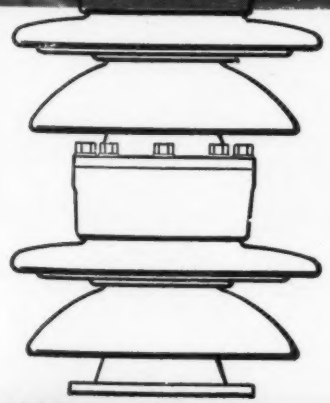

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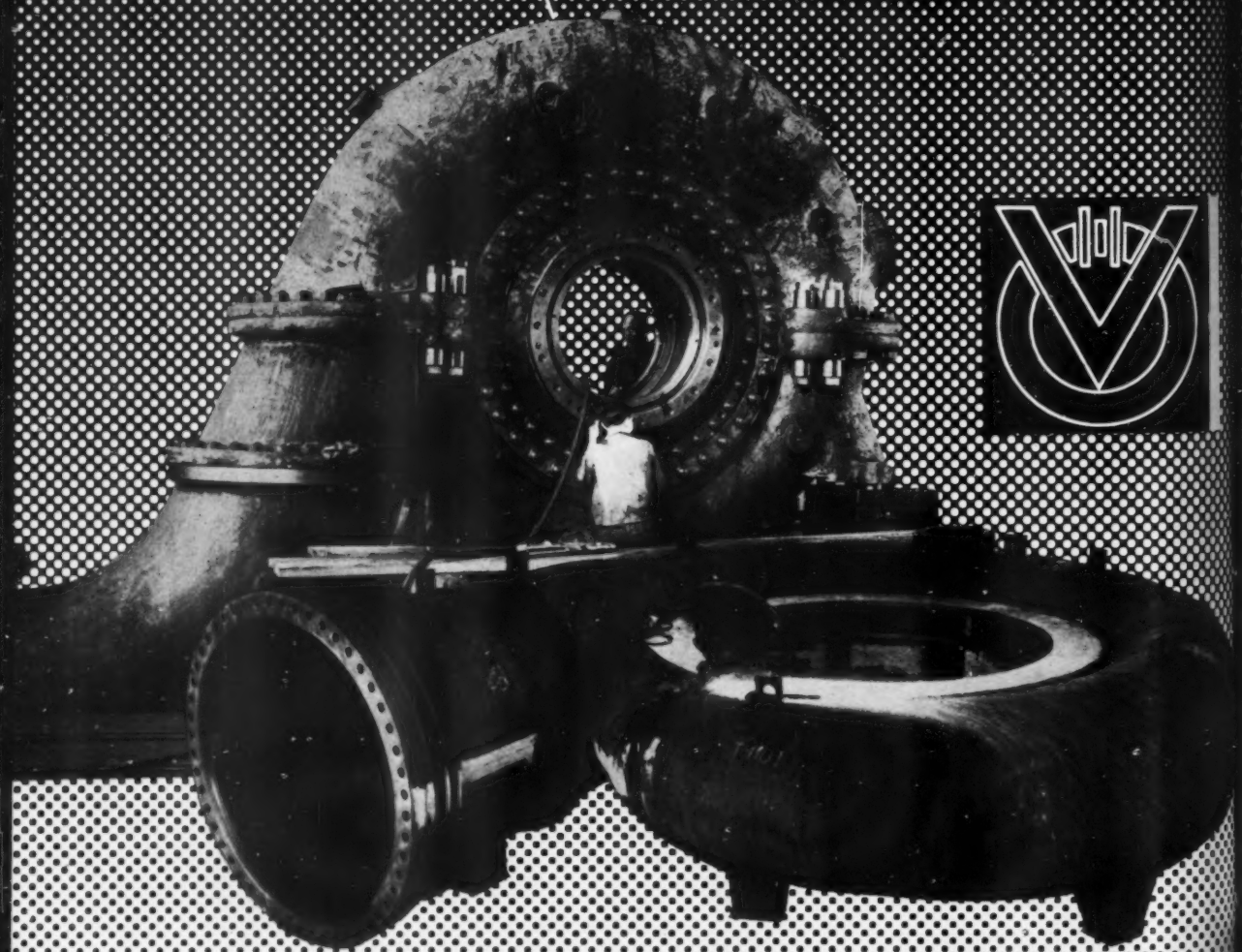

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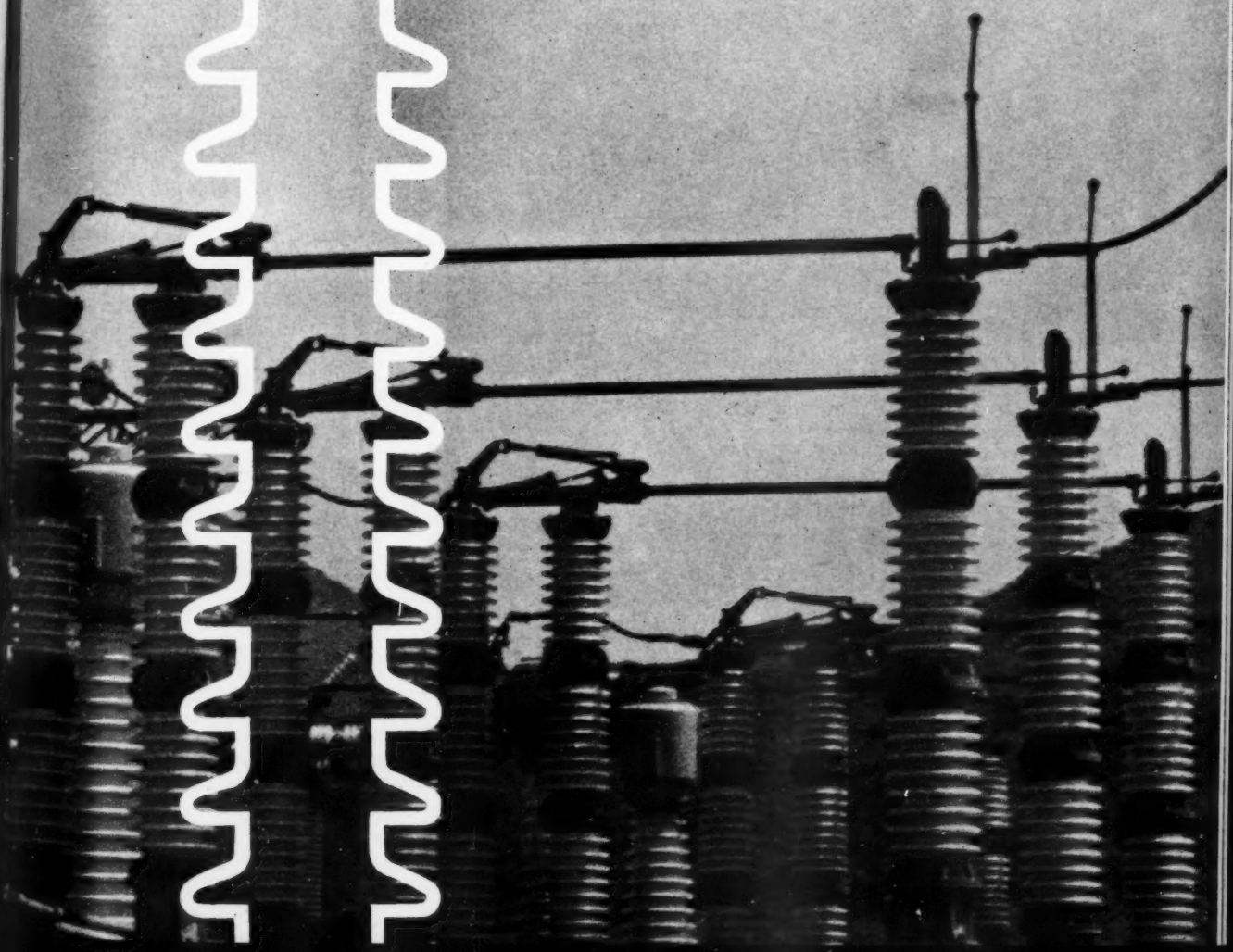
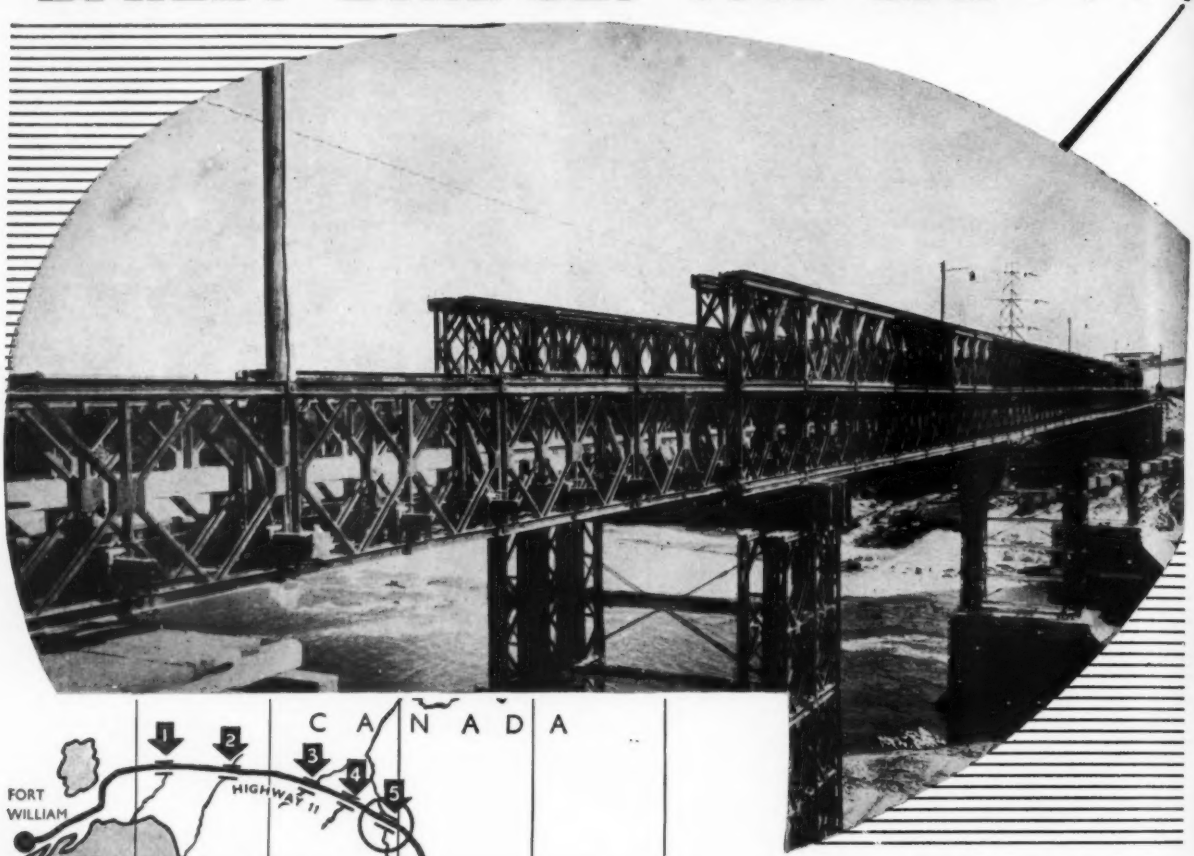
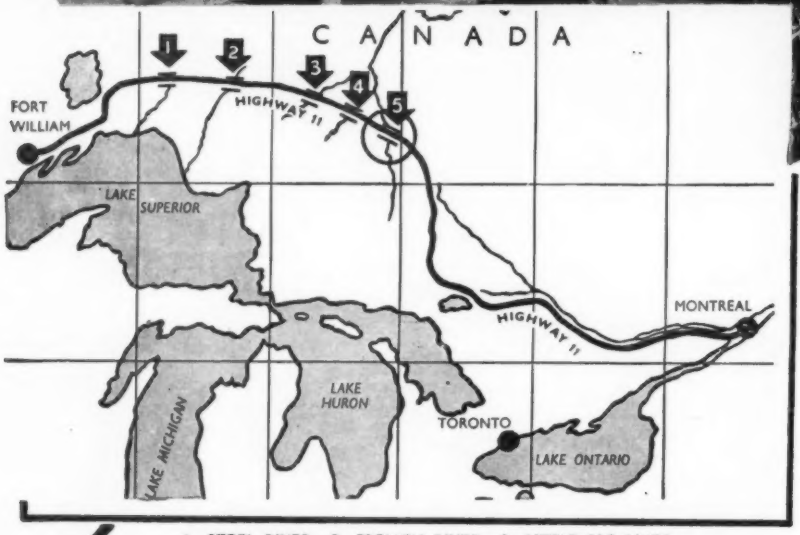


BAILEY BRIDGES THE GAP. . . .





1. STEEL RIVER. 2. FRENCH RIVER. 3. LITTLE PIC RIVER.
4. KAPUSKANG RIVER. 5. MATTAGAMI RIVER.

PHOTO. BY COURTESY: THE DEPARTMENT OF HIGHWAYS—ONTARIO.

The Bailey Bridge—550 ft. long—across the Mattagami River, is just one of the many in constant use by a large volume of traffic on Highway II—part of the Trans-Canada Highway.

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Full information on this or any other project requiring Bailey Bridging is readily available, from:

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The picture shows the assembly of an alternator 72,000 kVA, 600 r.p.m., for the Sils power station (Switzerland)

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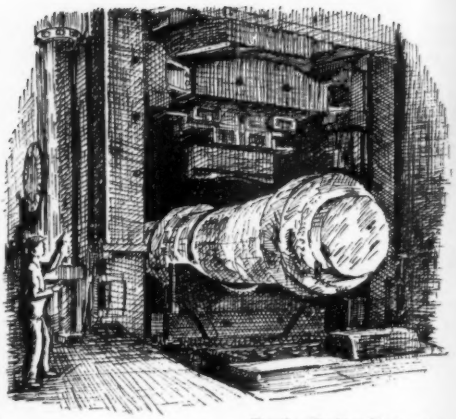
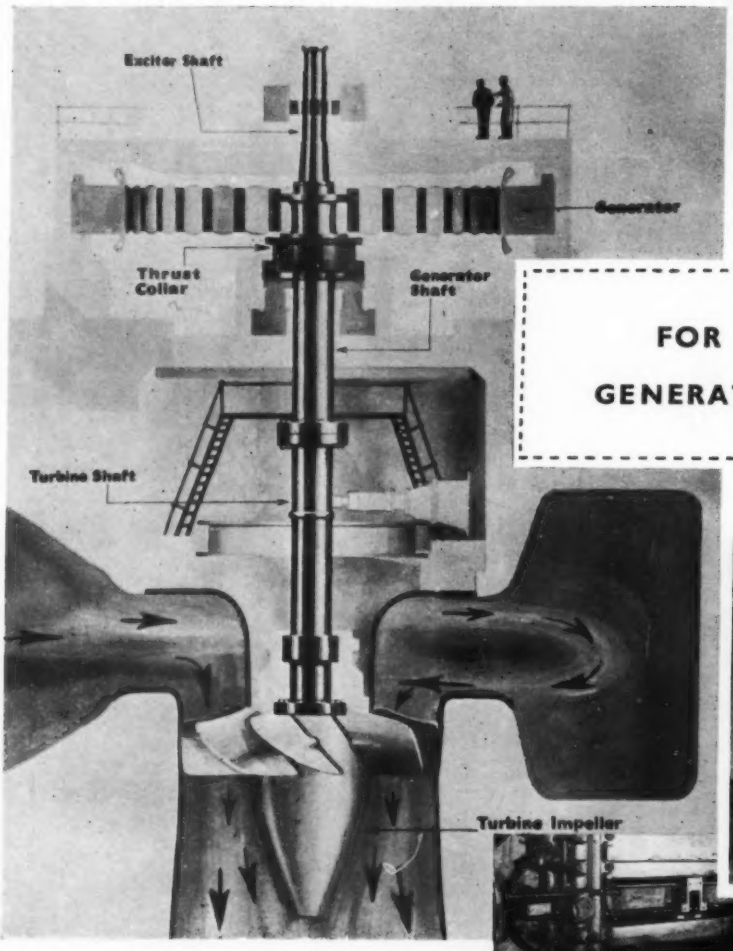
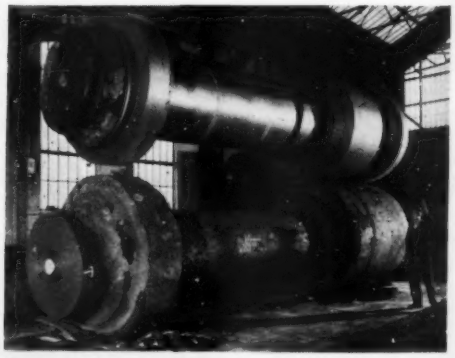
Steel Forgings...

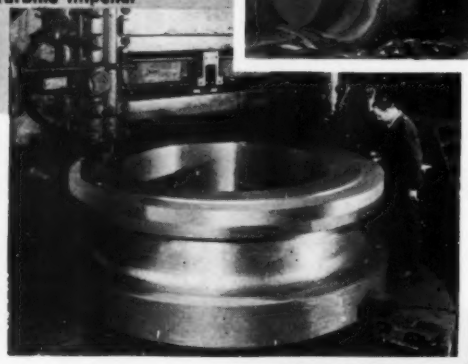


Forging Turbine Shaft under 7,000 ton Electro-hydraulic Press.



FOR HYDRO-ELECTRIC GENERATOR INSTALLATIONS

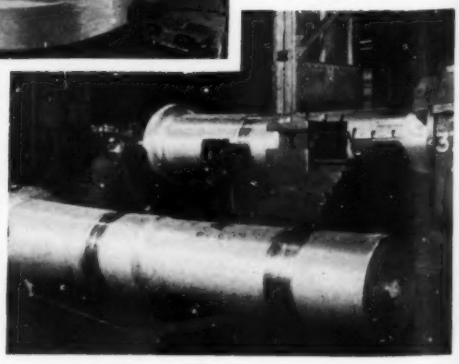




Rough machined and black forged steel Turbine Shafts, 22′ 11″ long.

Machining forged steel Thrust Collar, weight 18½ tons.

Forged steel Exciter Shafts, each weighing 8½ tons.



. . . in our high quality carbon and alloy steels — single piece engineering units up to 175 tons produced from ingots weighing up to 300 tons.

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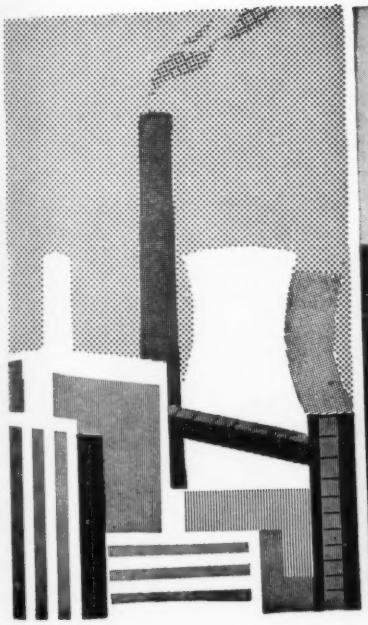

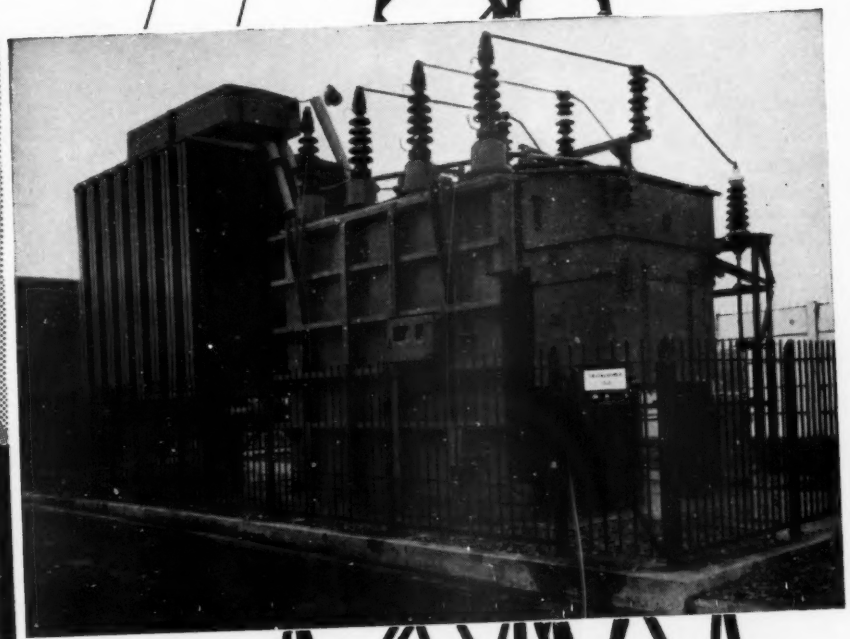

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(see photo and Voith pamphlet 1330e)

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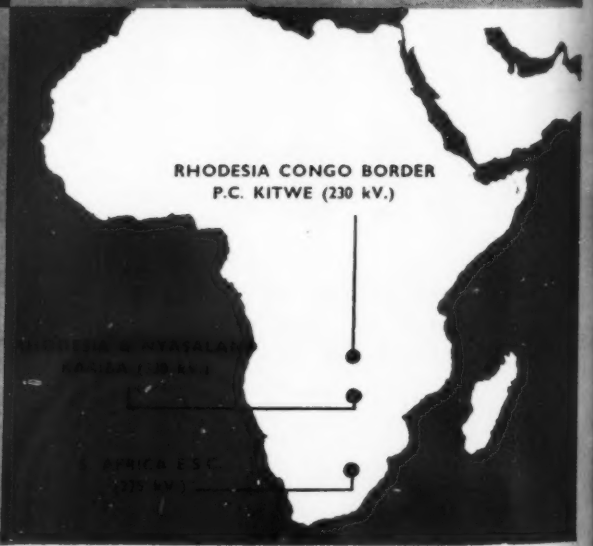

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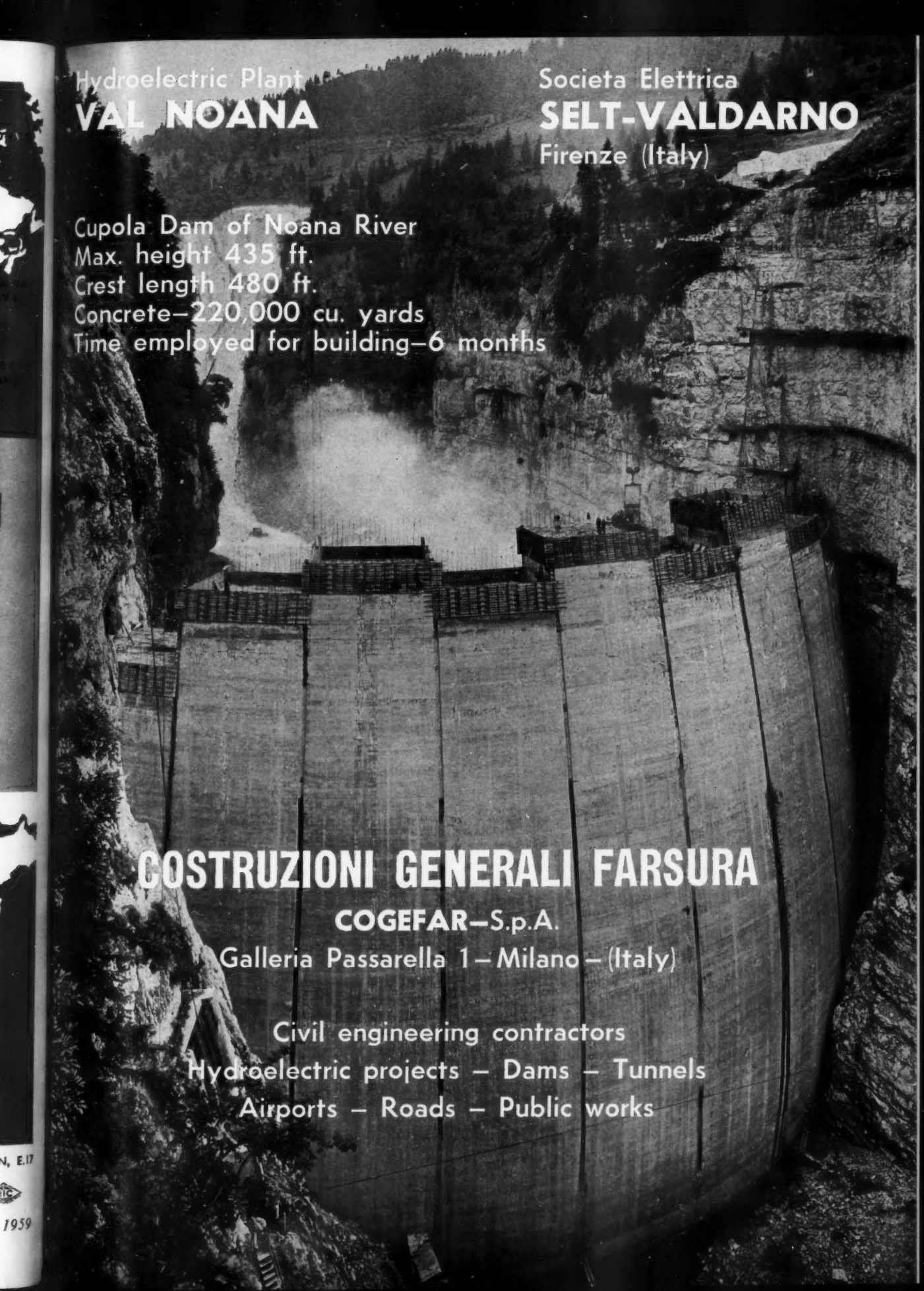
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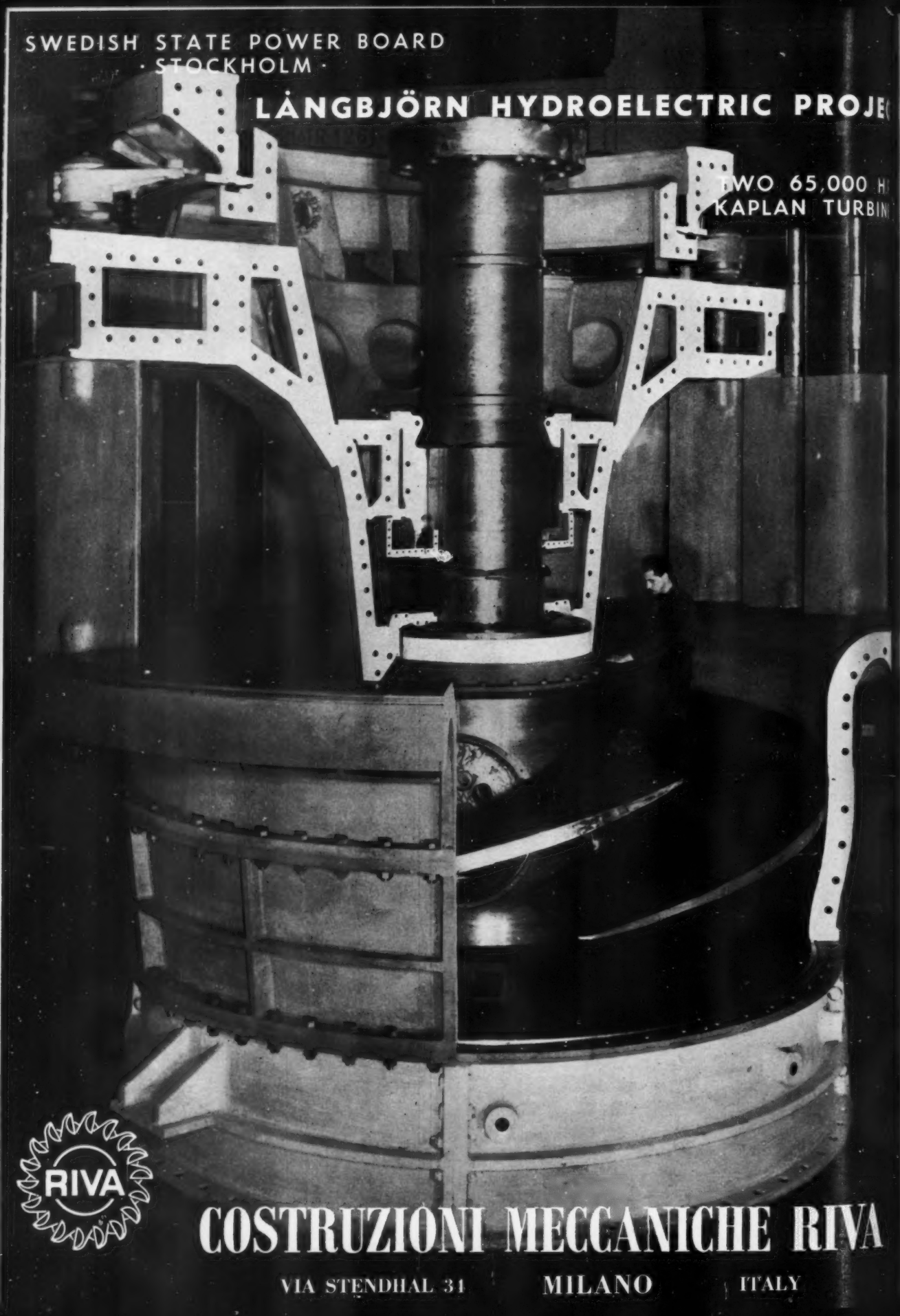
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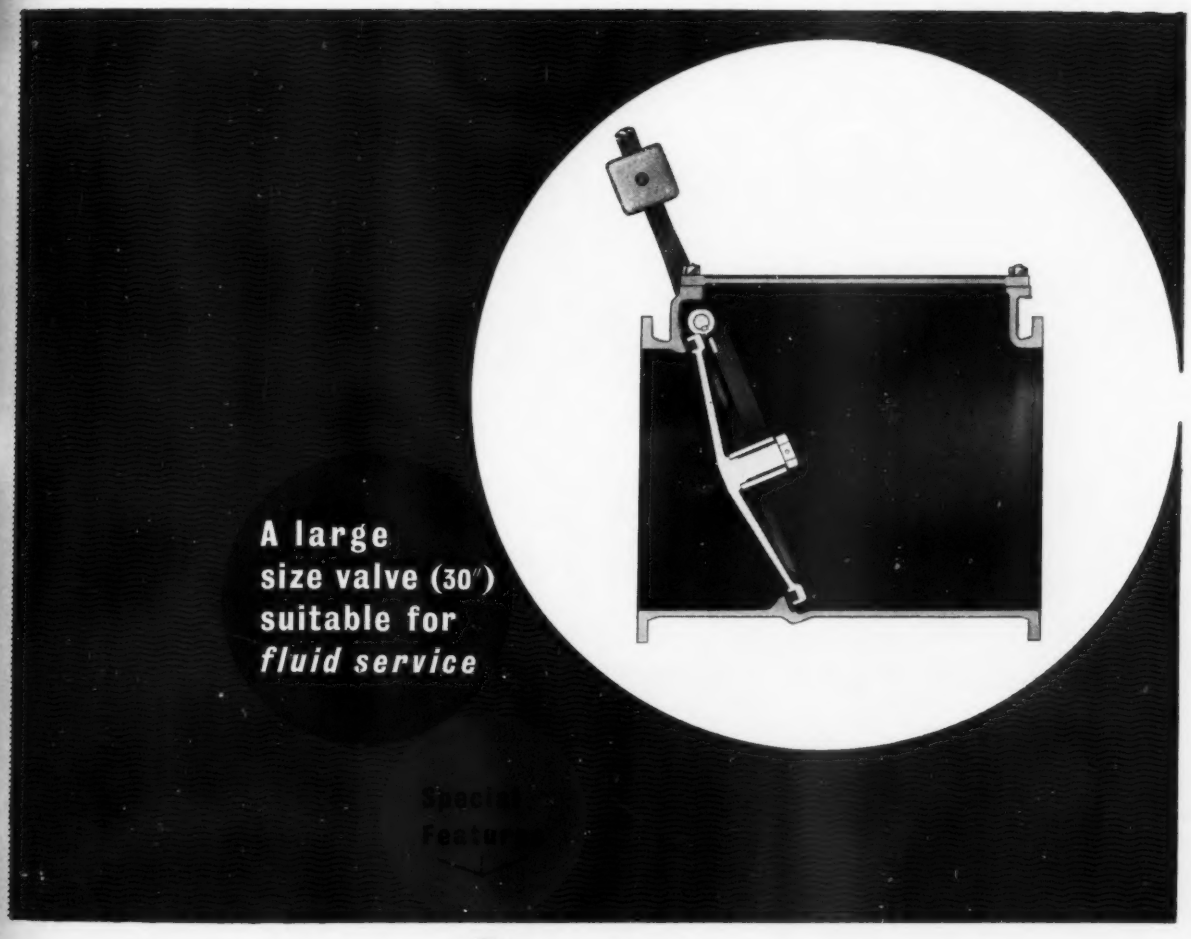

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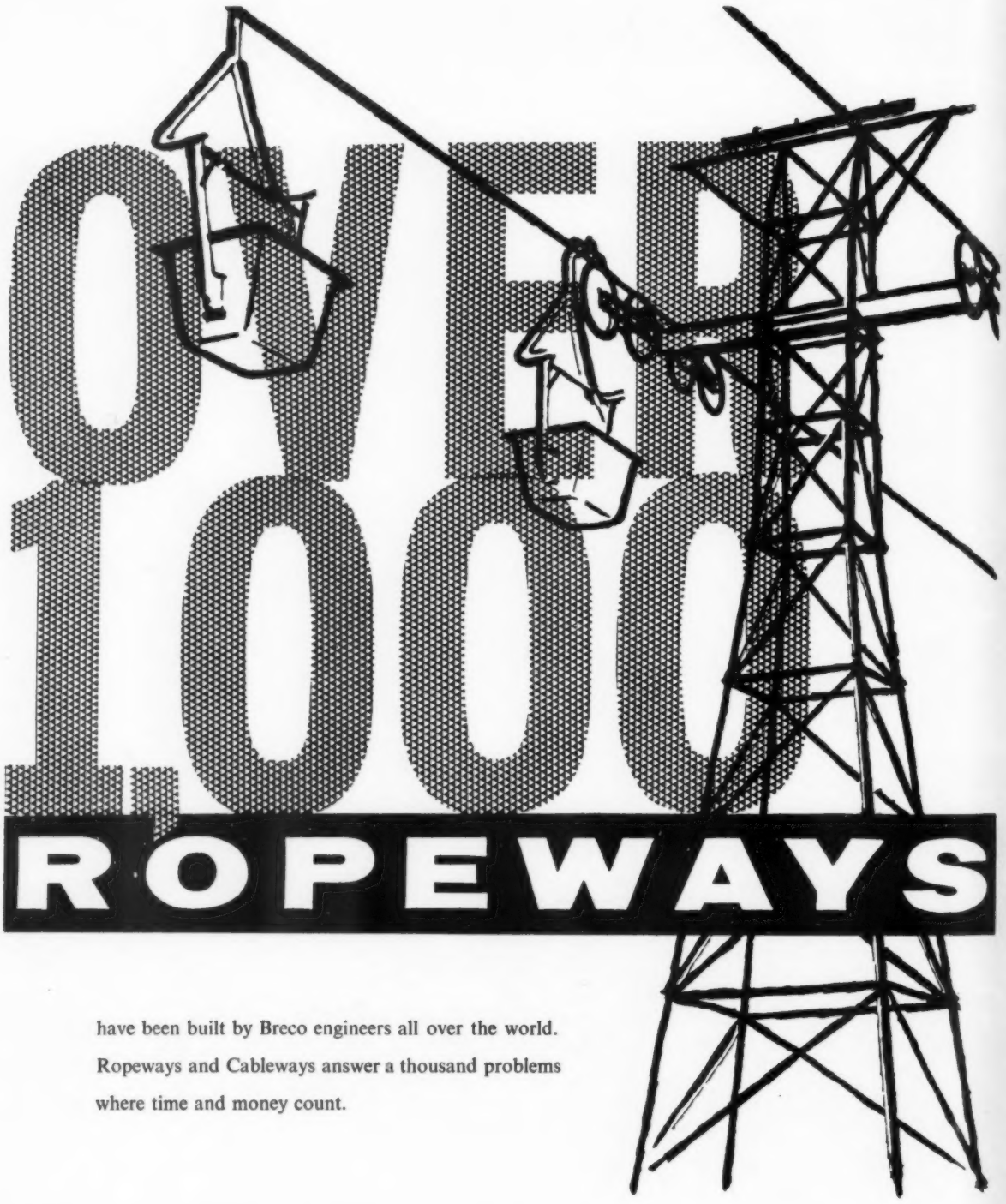

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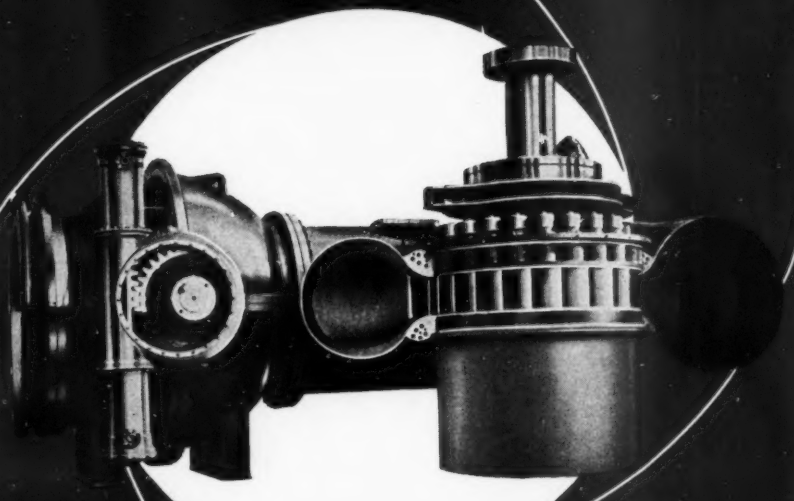

Turbines of the Francis, Kaplan and Pelton type for various heads and outputs, transformers, generators, outdoor and indoor switch-gear, control-rooms, etc.

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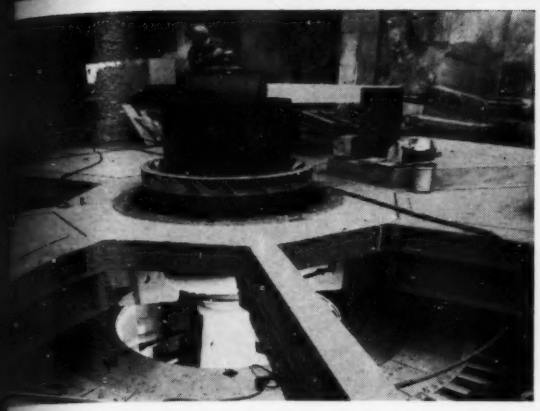
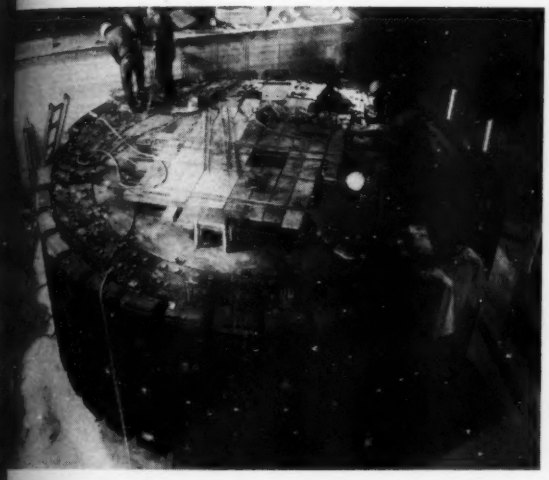
450 feet underground, this powerhouse is the "heart" of the great Alcan power development at Chute-des-Passes. This view taken before generators were erected, shows why millions of cubic yards of rock had to be excavated for this project. When installation is complete, this project will have a nominal capacity of 825,000 kva (one million horsepower) produced by 5 huge hydro-electric generators—the largest in the world—designed and built by C.G.E. at Peterborough.

CANADIAN GENERAL ELECTRIC



A C-G-E technician installs the main exciter of a generator which is reaching completion. Through the top cover openings one can see the stator winding connections and the completed rotor. The rotor weighs 734,000 pounds. Note also the ease of accessibility to the generator when the top covers are removed.

The stator and its completed winding have been accurately centred and the vital member (C-G-E pre-compressed spring-supported thrust and guide bearing) carefully installed. The stub shaft — used only as a support for the main exciter and its top end as an upper guide bearing journal—is being bolted to the generator driving disc.



Here rotor pole tapered keys are being driven "home" into the T-dovetails. This arrangement permits each pole to be driven solidly against its seat without applying force to its face. Before this operation, the 36 poles (weighing 196,000 pounds) were carefully fitted and centred to the rotor rim.

Here rotor rim laminations are being keyed and piled around the rotor spider. When completed, the rim will contain over 15,000 laminations. Accurate distribution is essential to produce a balanced rim of uniform thickness. Note the radial rotor rim ventilation ducts which ensure uniform generator cooling—a patented C-G-E feature.

1,000,000 HORSEPOWER OF C-G-E GENERATORS INCREASE ALCAN'S SAGUENAY POWER SYSTEM BY 40%

Three hundred miles north of Quebec City, the first power is flowing from generators at Chute-des-Passes . . . generators designed and built by Canadian General Electric at Peterborough, Ontario, to help harness more of Canada's water resources.

A select crew of highly-skilled C-G-E technicians is completing the installation of five huge hydro-electric generators in the underground power-house—heart of this great new power project for the Aluminum Company of Canada Limited. Each generator has a nominal rating of 165,000 kva, the largest hydro-electric units in the world.

The choice of C-G-E Generators for Chute-des-Passes, on the Peribonka River, is a further indication of the confidence that progressive Canadian industry has in our products and services. These are available to *you* also.

Whether your particular requirements are large or small, C.G.E. has the facilities, technical knowledge, and experienced personnel to serve all your electrical needs—call the C-G-E office nearest you, or write to Apparatus Dept., 107 Park Street North, Peterborough, Ontario.

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Bucyrus-Erie excavators

are preferred for major jobs

On public works construction as well as quarrying and mining, Bucyrus-Erie heavy-duty shovels and draglines are preferred. Illustrated is a 110-B 4½ cubic yard shovel, one of three employed on the Serre-Poncon earth dam project on the Durance river in Southern France. Two 150-B draglines were engaged on the same scheme. Over a hundred years have gone by since dam projects in this region were first visualised. Only the development of present-day technical resources has made the project possible—and the choice of excavators? Bucyrus-Erie.

BUCYRUS ERIE

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South Milwaukee, Wisconsin, U.S.A.

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2000 Water Turbines

have been manufactured by Tampella so far. The Turbines numbers 2000 and 2001 were delivered to the Soviet Union for the Kaitakoski Power Plant where they were recently taken into operation. The delivery comprised two Kaplan-turbines with the following technical data:

H = 8.5 m.
Q = 100 m^3/sec.
N = 10,050 H.P.
n = 100 R.P.M.

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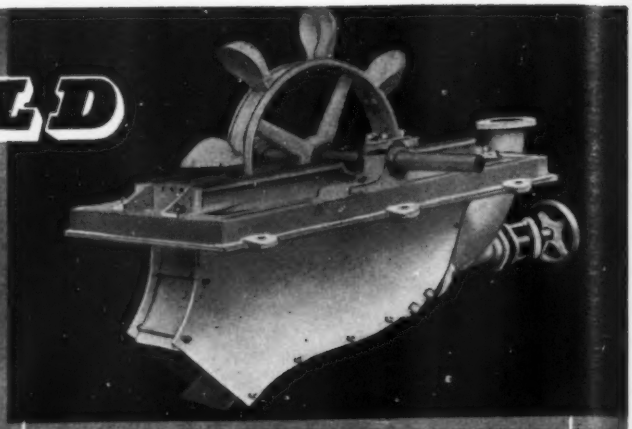
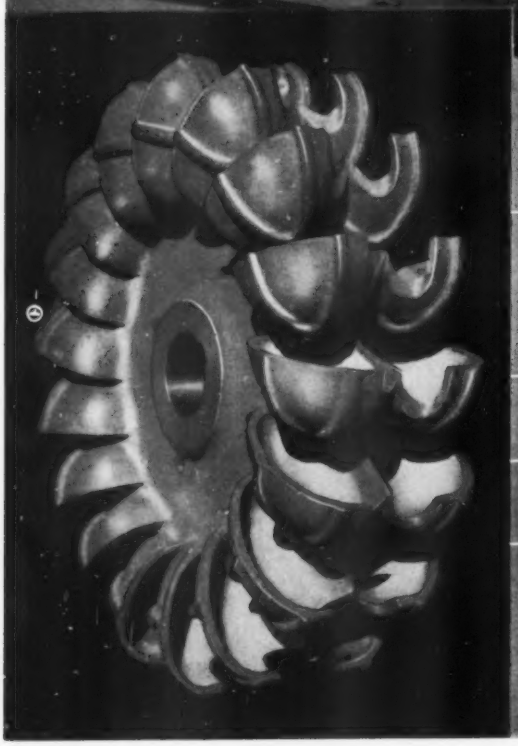
FROM 2H.P. TO 2000H.P.



Are backed by 60 years of development



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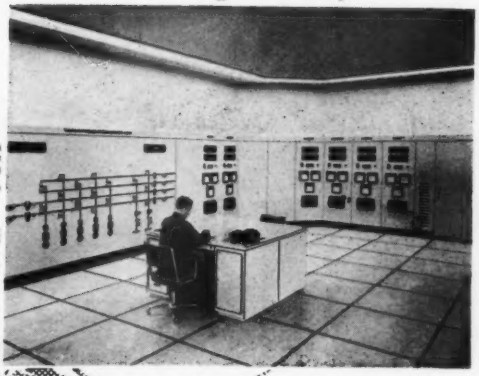
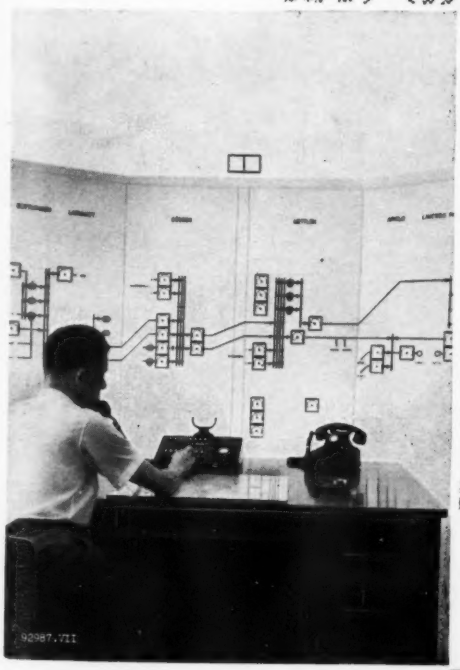


TELECOMMUNICATION



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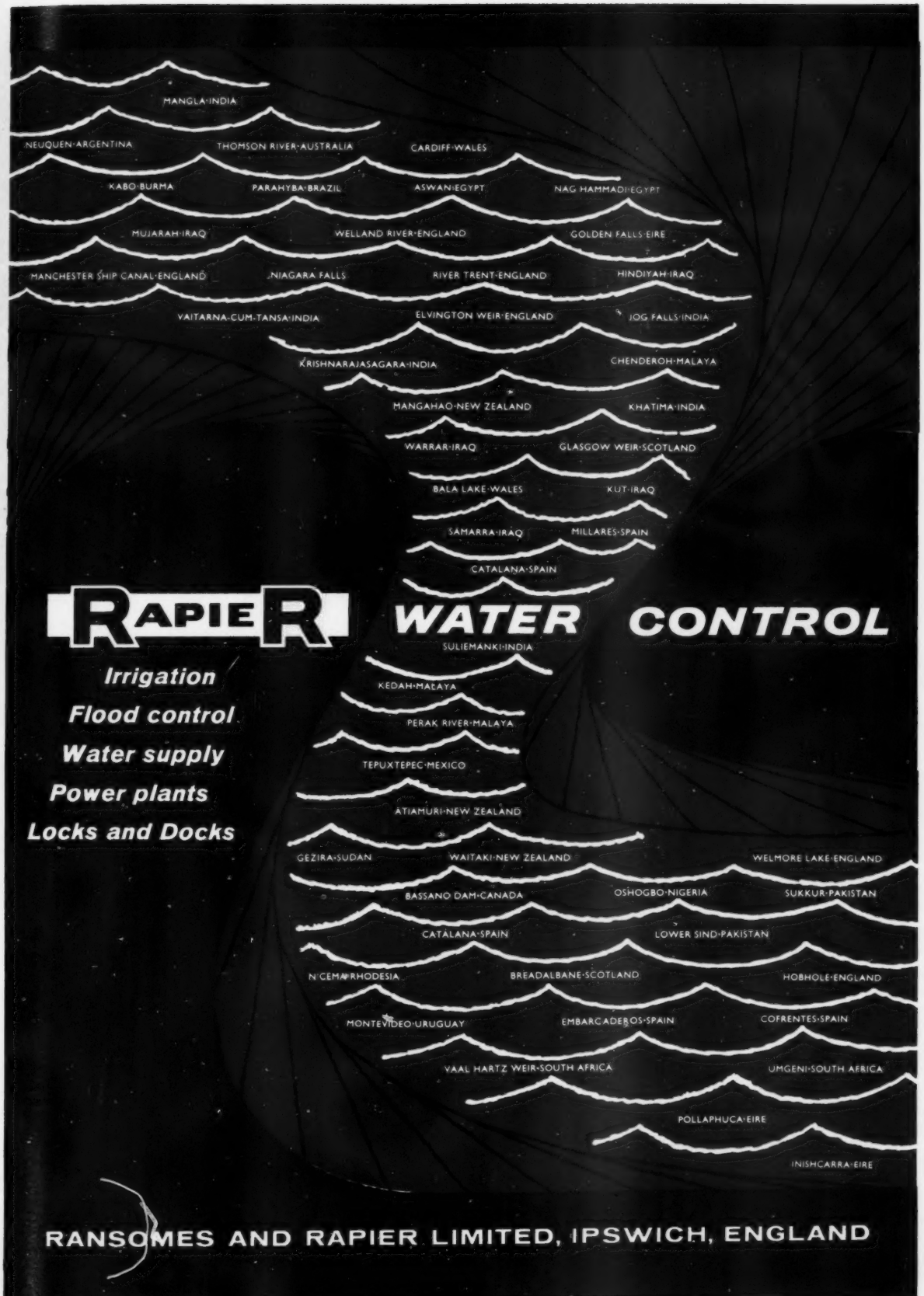
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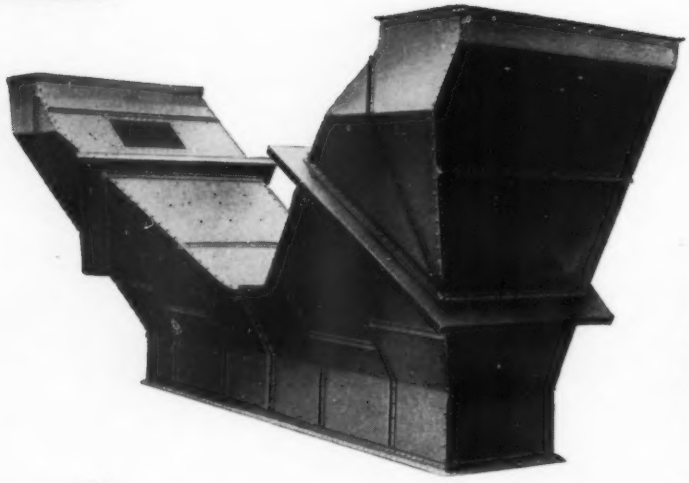

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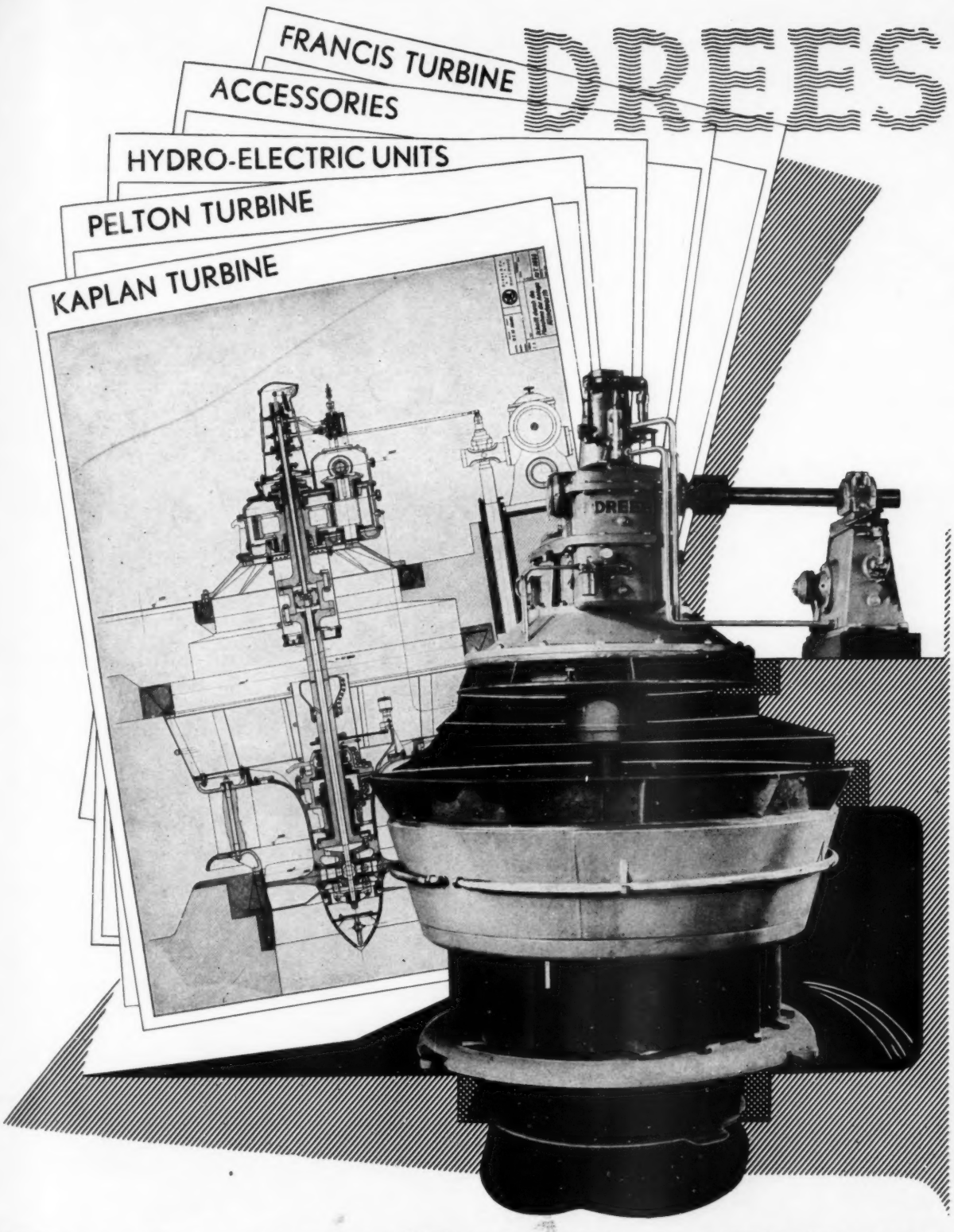

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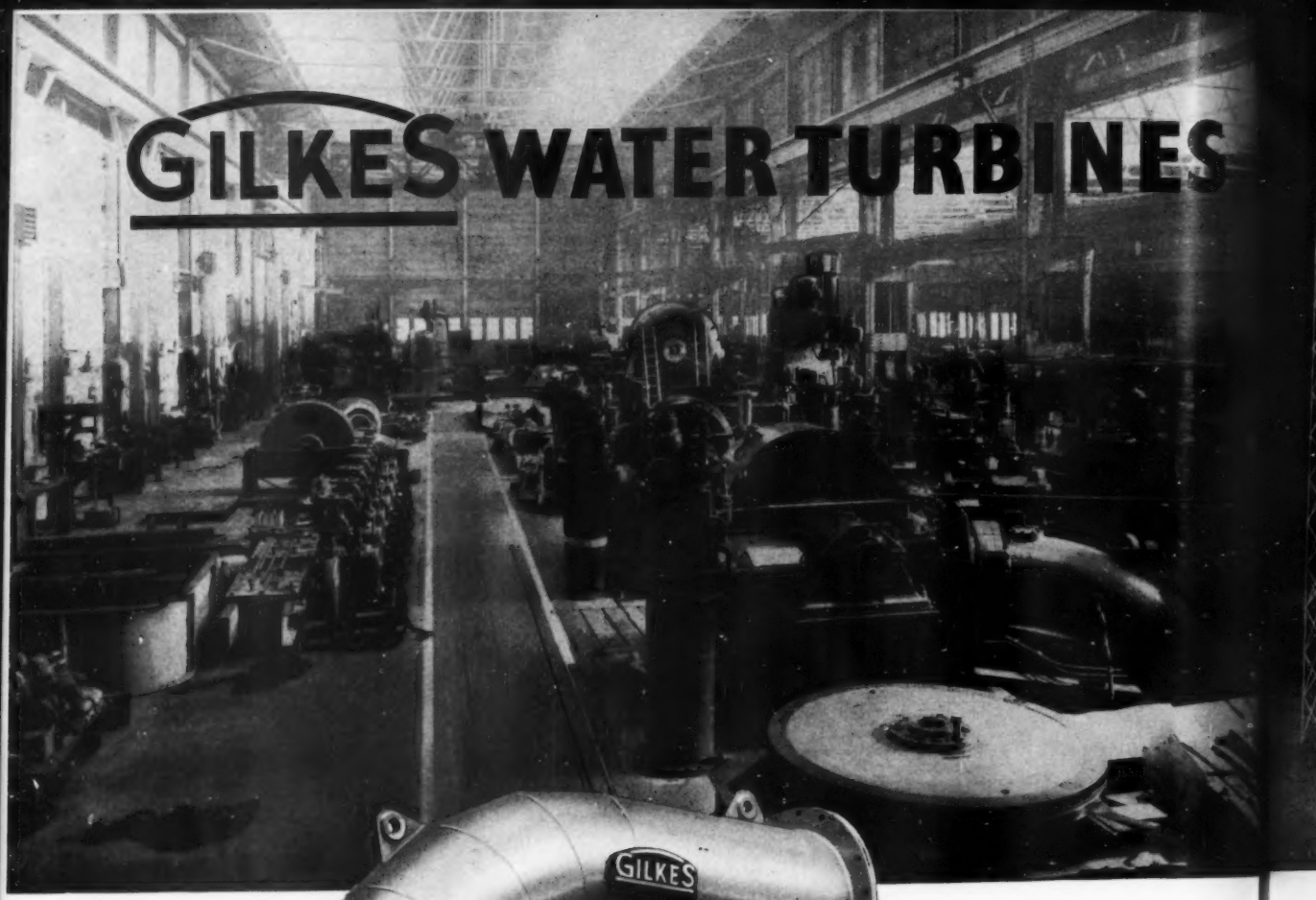

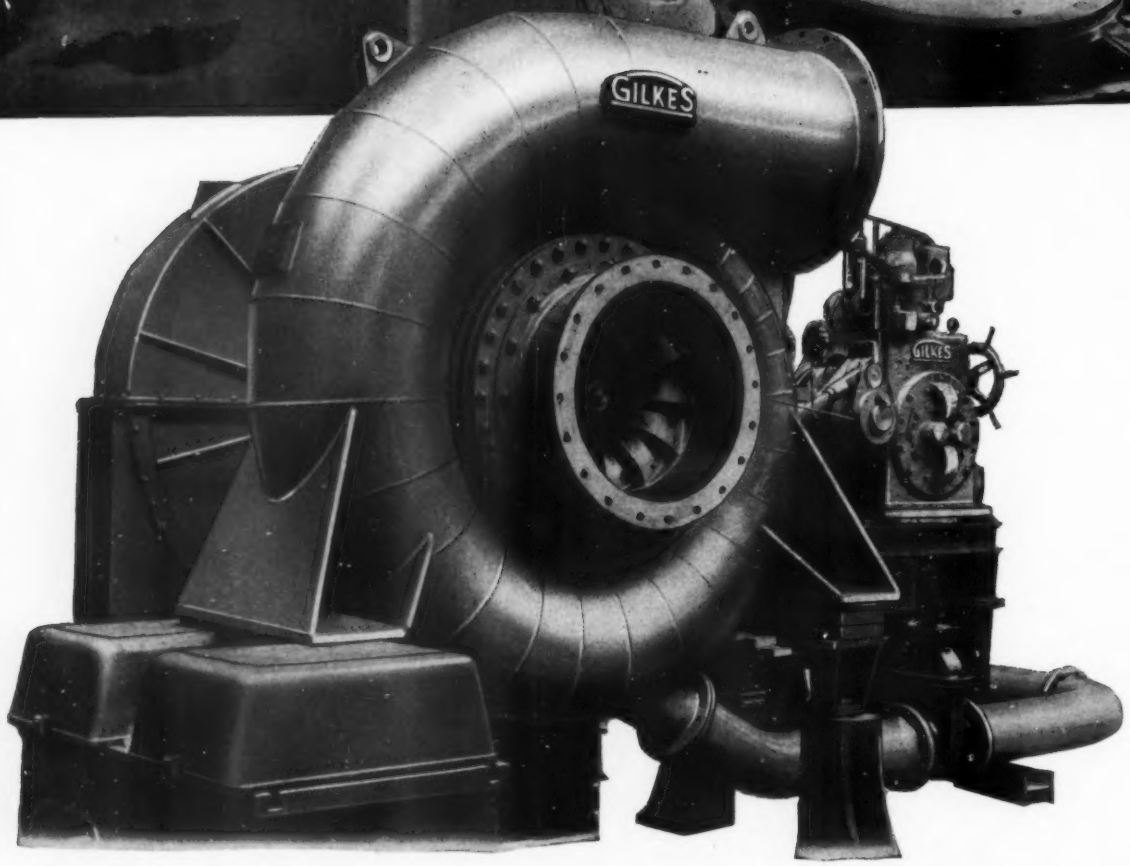

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General Electric's extra high voltage towers will be 160 feet high and 180 feet wide. Model towers shown above are built to 1/25 scale. A model of an ordinary tower is also shown.

The world's largest bushings, for 650-kv power transformers, will be used in G. E.'s prototype EHV system.

For long-range power planning...

GENERAL ELECTRIC ANNOUNCES PROJECT EHV—A NEW CONCEPT IN SUPER VOLTAGE POWER TRANSMISSION

Here's news from General Electric that could affect long-range plans for power development in many parts of the world. It's project EHV—Extra High Voltage. Right now, General Electric is building a system that will transmit electricity at voltages higher than ever before. The system is scheduled to carry power at a maximum of three-quarters of a million volts—higher than any existing power line in the world. Later the line may be altered to send up to a million volt power.

The entire electrical power industry is expected to benefit from this project. In the future, more and more of the world's great cities will be able to "bring in" low cost power from distant water power sites. This will not only speed economic growth in many countries, but it will also help conserve precious fuel supplies.

Project EHV is a five-year program designed to provide actual operating experience and other valuable information on extra-high voltage transmission. It is another in a long series of General Electric developments that have earned G. E. its reputation for designing and manufacturing reliable products for utilities throughout the world. *International General Electric Company, Dept. 59-5E, 150 East 42nd Street, New York 17, N.Y., U.S.A.*

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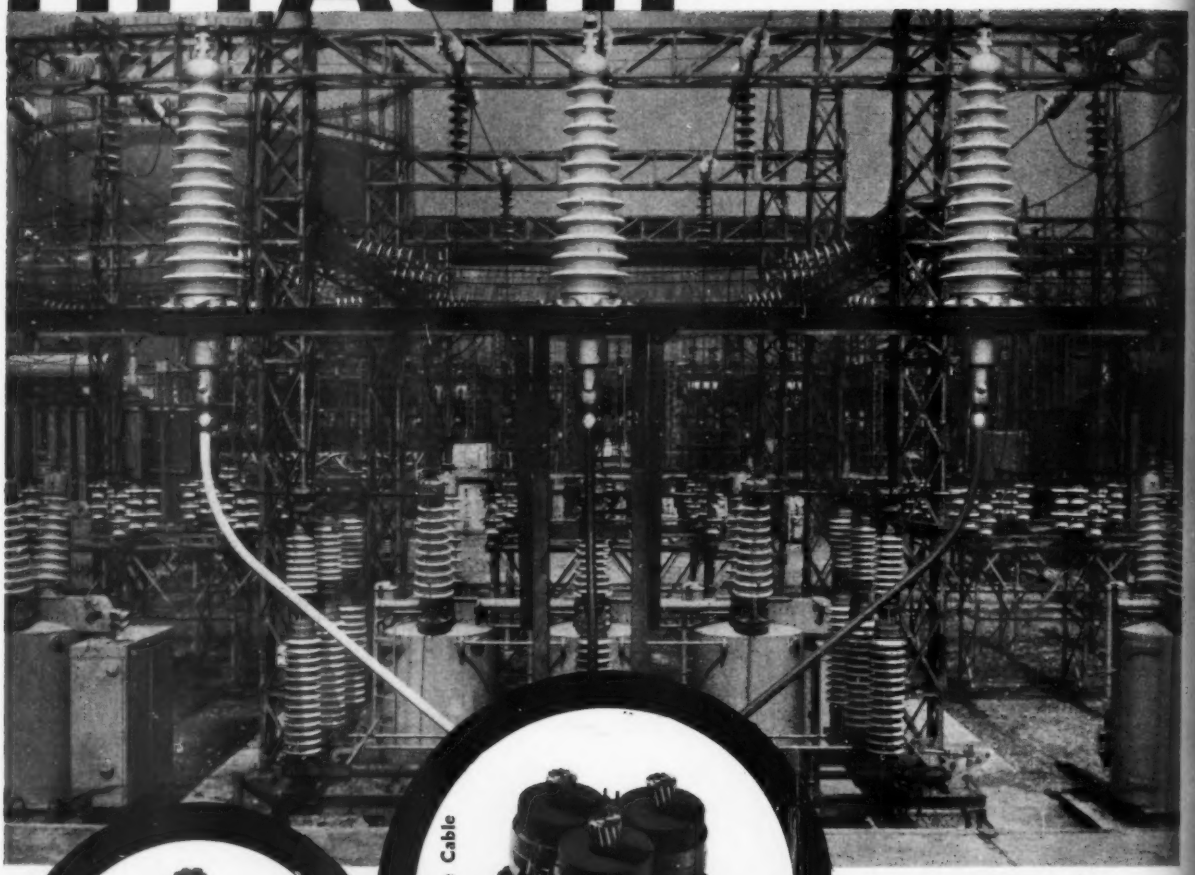

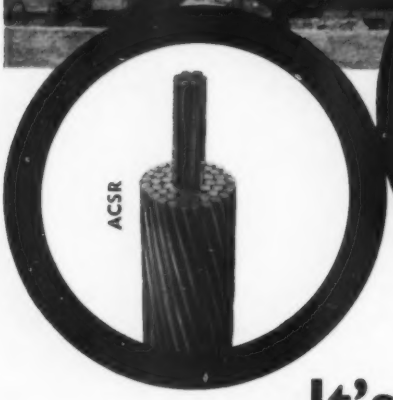

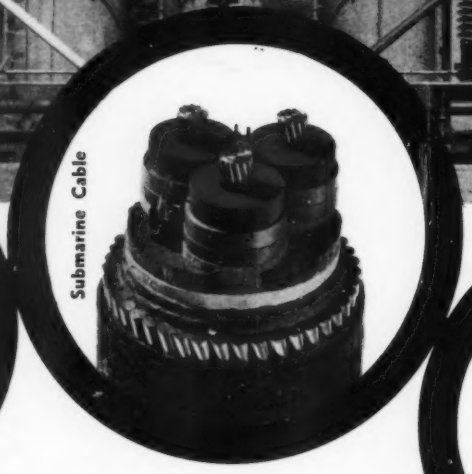

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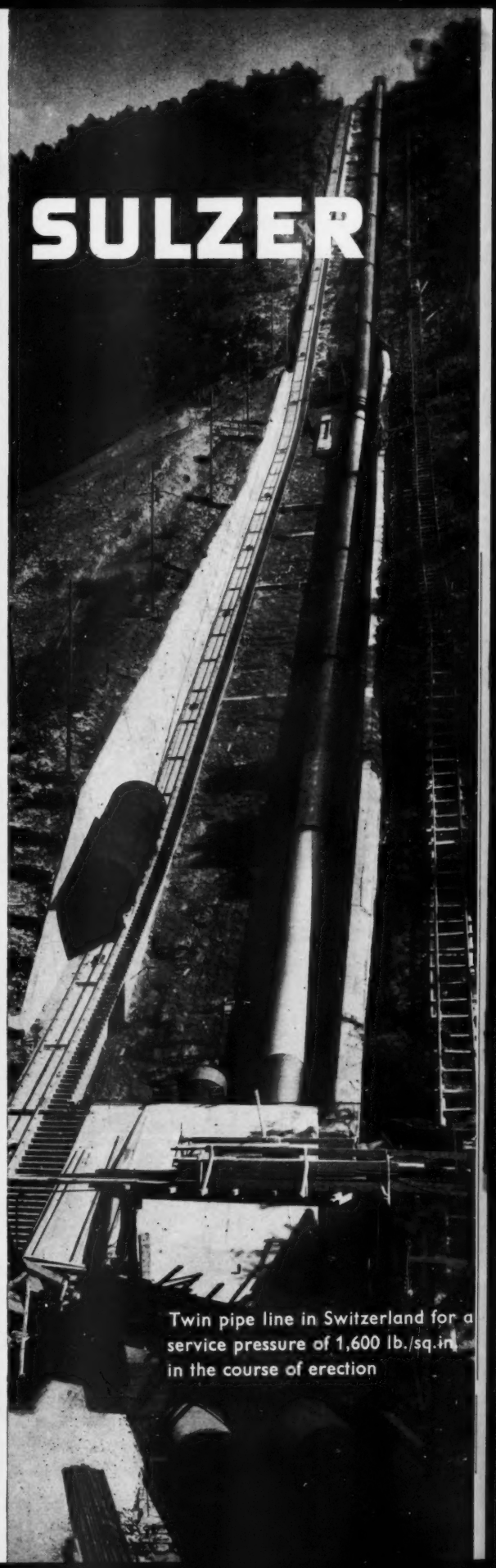
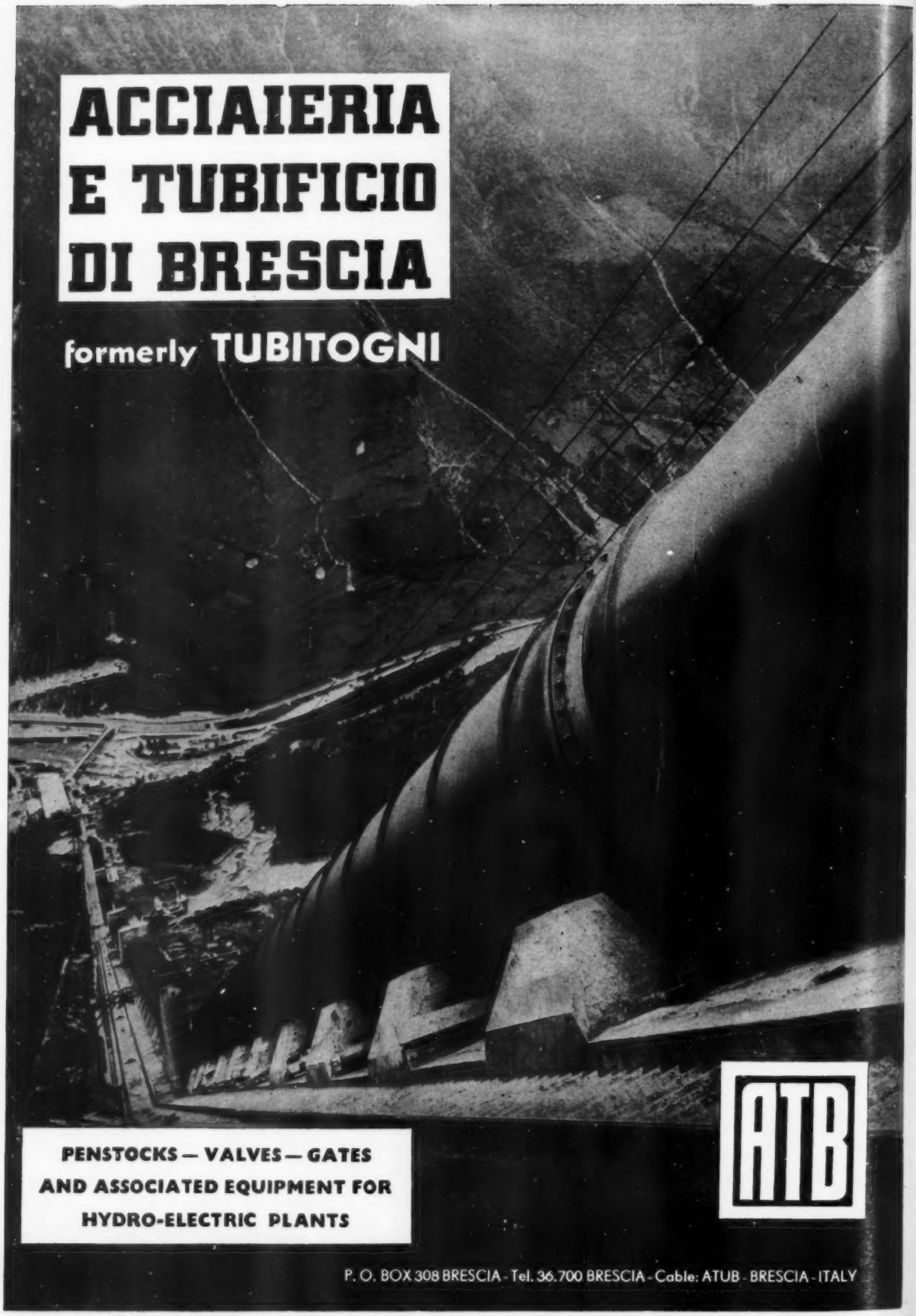
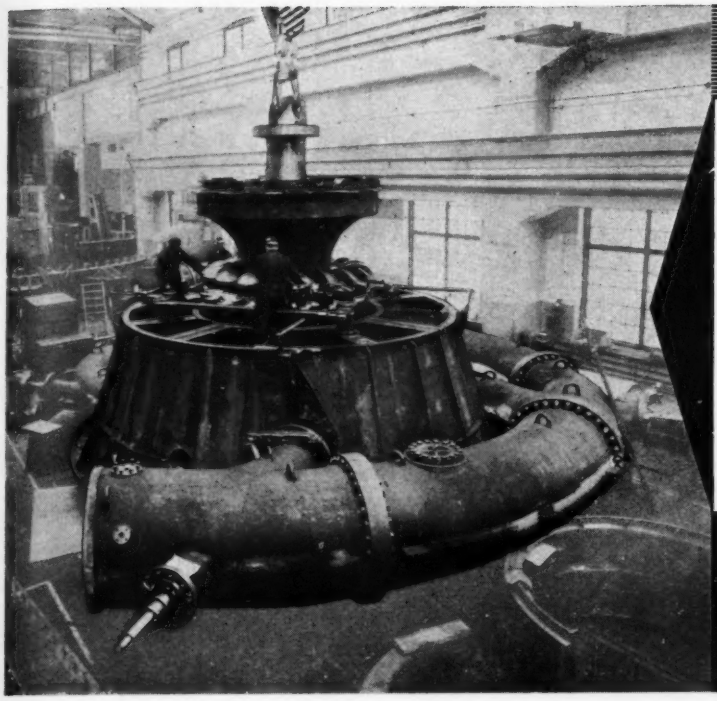


Twin pipe line in Switzerland for a service pressure of 1,600 lb./sq.in. in the course of erection



United Kingdom Representatives: L. A. Poole & Co., Clun House, 17, Surrey Street, Strand, London, W.C.2. Tel. Covent Garden 0021/2

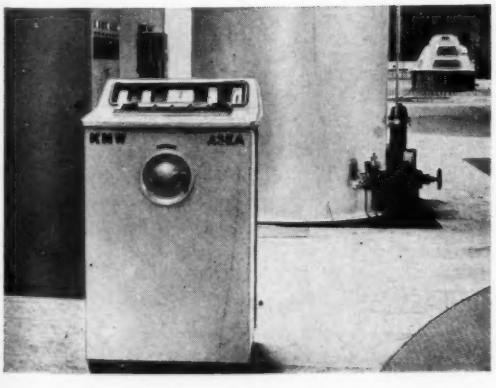


The Largest Vertical

PELTON Turbine

in Norway is a KMW Turbine

In the last half of 1958 the first of the two Pelton Turbines ordered from KMW by the Hjartdöla Power Station in Norway was put into operation; the second unit will be put into service shortly. The turbine has a nominal output of 71,000 H.P. at 555 m (1,820 ft.) net head and 428 r.p.m.



Both the turbines are equipped with the ASEA-KMW system of electro-hydraulic governing. Impulses from the electronic regulator are converted into mechanical motion in the electro-hydraulic actuator. This motion is transmitted via a separate servomotor to the deflectors and the control valves of the needle servomotors, which are linked by a conventional system of cams to provide the correct combination of controls.

This is the first time that an ASEA-KMW electro-hydraulic actuator has been used for a Pelton turbine. When the first unit was started up and put into service, the regulating system operated perfectly without any adjustment. Alterations in speed when throwing-off load were very small and the return to normal revolutions was remarkably quick.

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Diameter 2·25 to 1·60 metres, Power 200,000 H.P.

GRANGENT DAM on the Loire
One of the four Crest Gates, Sector Gate Type (12·5 m. x 14·4 m.) during erection.

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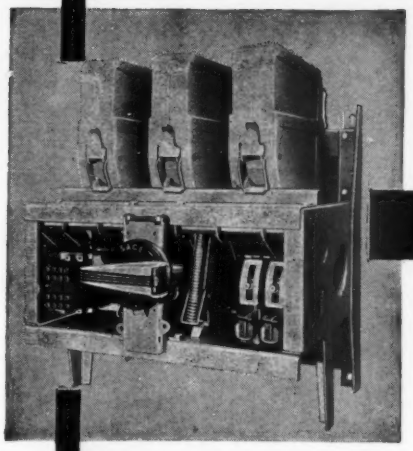

Circuit-Breaker type "P"
Rated voltage 600 V
Rated current up to 4,000 A
Breaking capacity up to 55 kA
at 500 V a.c.

Circuit-Breaker type "Z 2"
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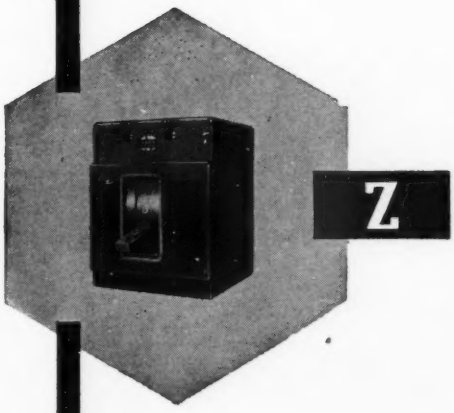
Rated current up to 600 A
Breaking capacity up to 20 kA
at 500 V a.c.



Circuit-Breaker type "Z"
Rated voltage 500 V
Rated current up to 150 A
Breaking capacity up to 12 kA
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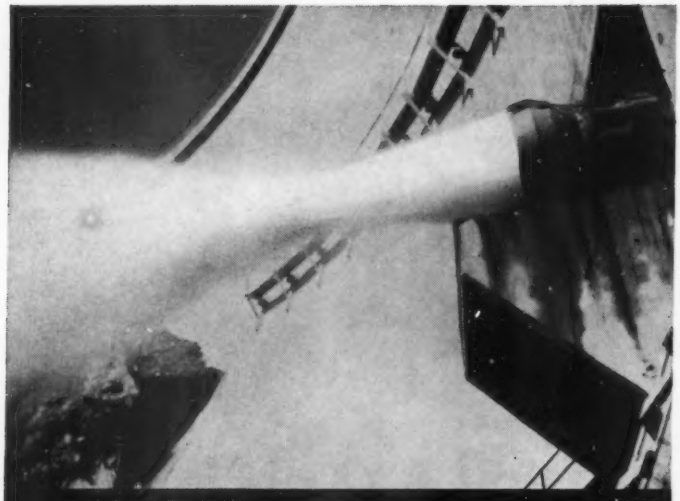
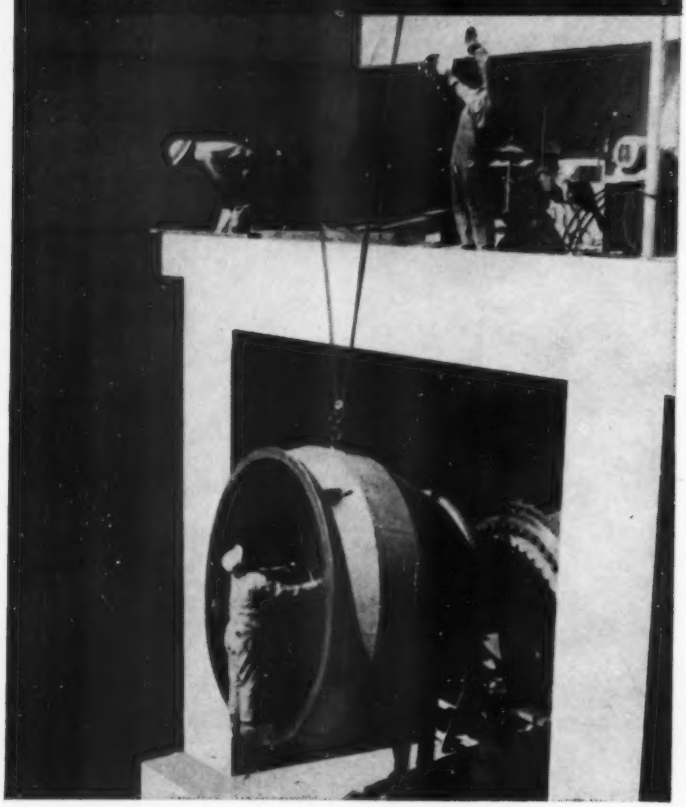


This 18-inch Ring-Jet Valve, installed by the City of Bremerton, Washington, is manually operated. Under a 115-foot design head, it discharges 76 MGD when fully opened. Note how the hood concentrates the discharge stream.

One of two 66-inch A-C Ring-Jet Valves during final assembly at Tulloch Dam, California. These valves are used for an irrigation bypass, operate under a 145-foot head.



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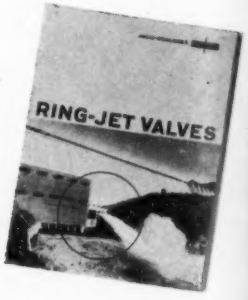
for easy regulation and control of water under free discharge

Allis-Chalmers Ring-Jet Valves are a logical development from the Howell-Bunger Valve. For use under high- and low-head, free-discharge conditions, the A-C Howell-Bunger creates a substantial amount of spray in providing maximum dispersion to minimize erosion. Now, in the A-C Ring-Jet Valve, the addition of a steel hood creates a concentrated jet, reduces spray, and helps dissipate the energy in the discharged water.

This new A-C Ring-Jet Valve gives you the simple construction, radially-balanced hydraulic design and easy operation of the A-C Howell-Bunger Valve, while very nearly matching its high discharge coefficient. Ring-Jet Valves are available with standard manual and/or motor-driven operators. Hydraulic operation can be provided, and is especially applicable to the larger sizes and higher head applications.

To obtain complete information on the A-C Ring-Jet Valve, contact your nearest Allis-Chalmers office, or write Allis-Chalmers, Hydraulic Division, York, Penna.



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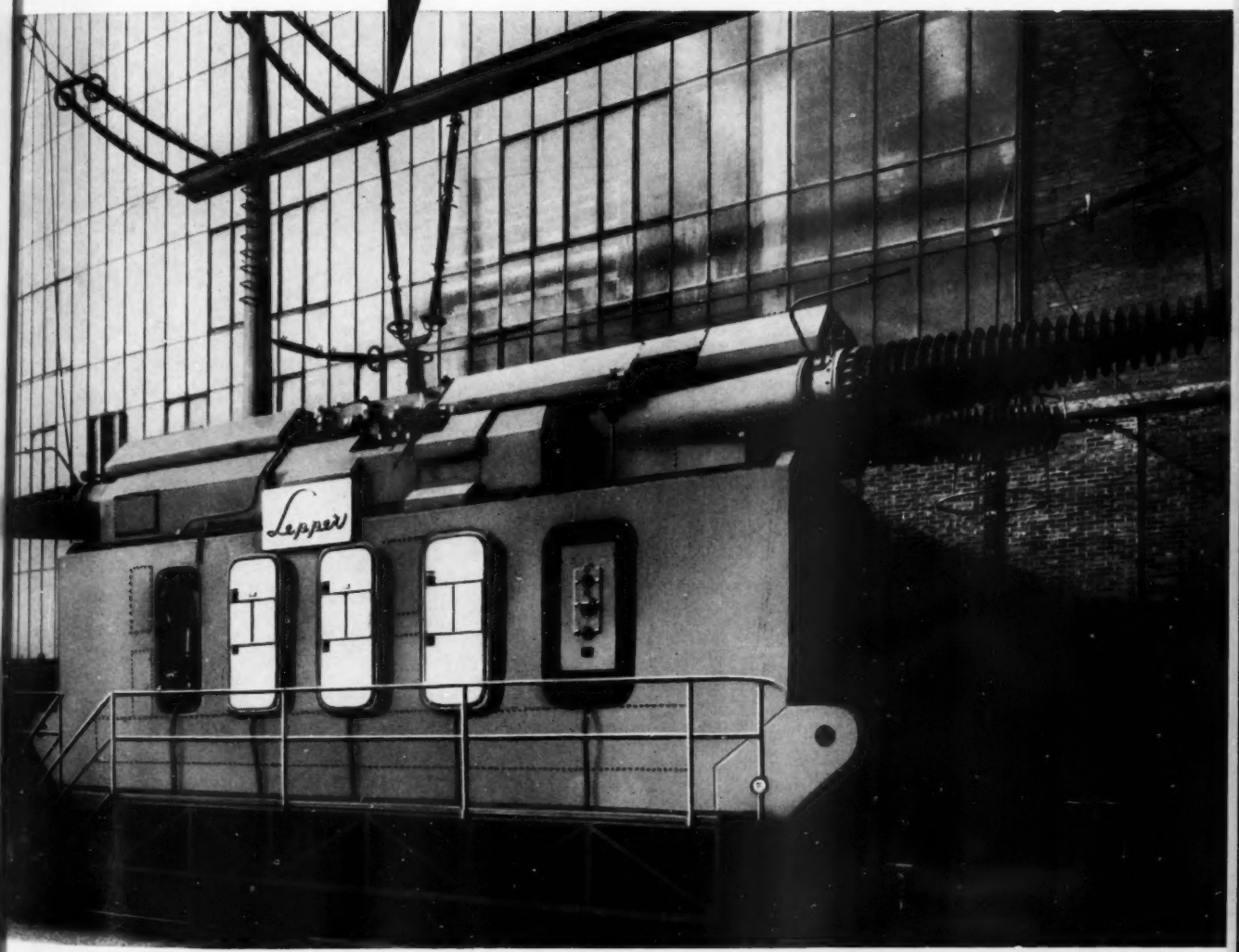

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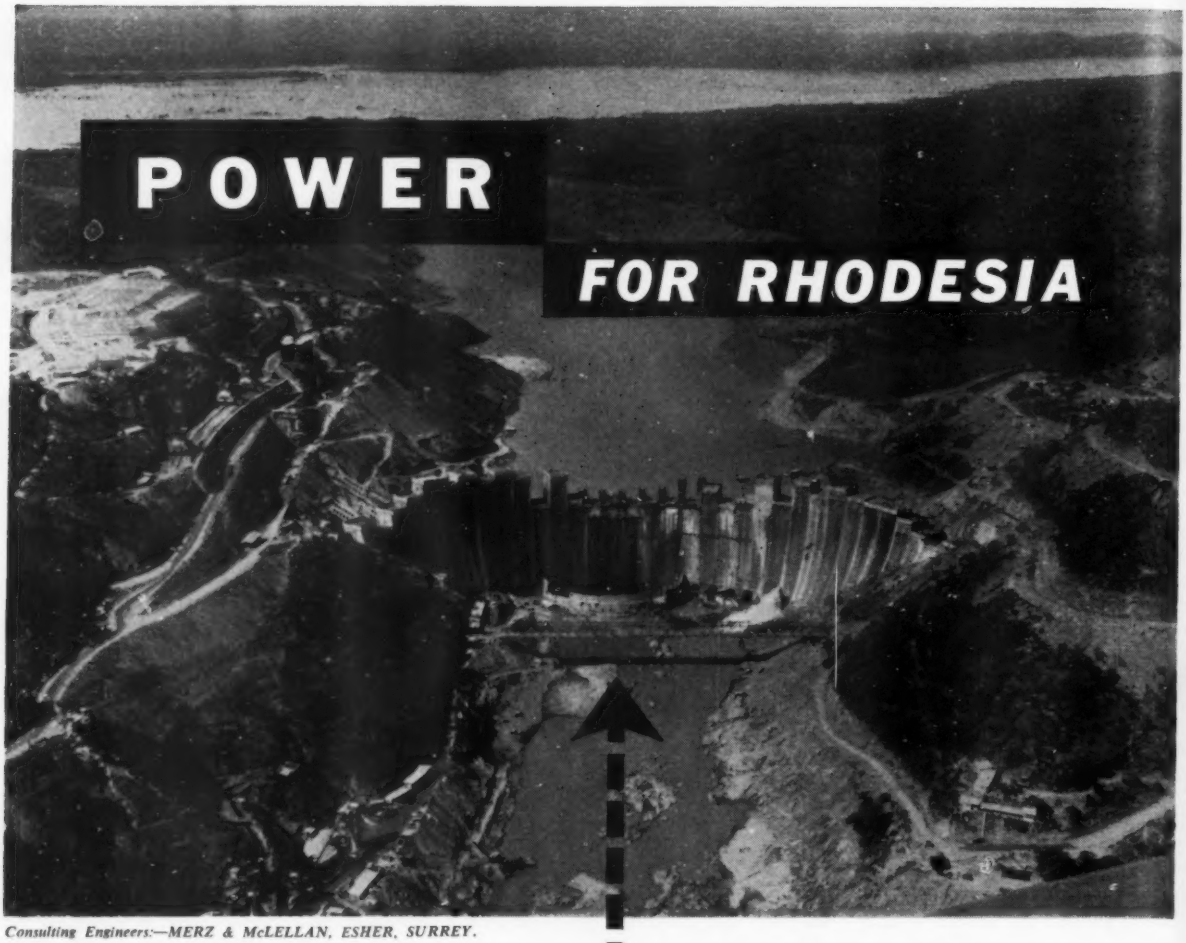
West Germany



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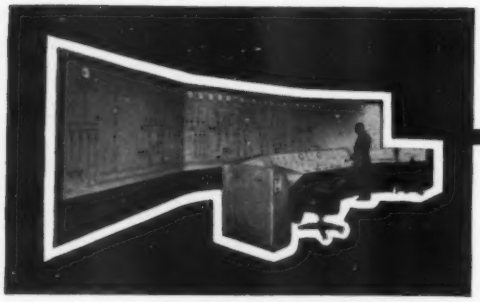
S.T.C. Telecommunication & Control Systems for the Kariba Hydro-Electric Project



The rising structure of the new Kariba Dam on the Zambezi River indicates the immensity of this undertaking which will eventually result in the routing of electric power at 330 000 volts to the 'copper belt' towns of Northern Rhodesia and the major cities of Southern Rhodesia.

S.T.C. are supplying and installing equipment which will control and supervise transmission lines involving a total length of 935 miles. This will include Voice Frequency Telegraph equipment, Remote Control and Remote Indication equipment, Remote Metering equipment, Teleprinter equipment, Photo-facsimile equipment and Telecommunication equipment. Power Line Carrier equipment is also being installed by S.T.C. whose unequalled experience in Remote Control and Indication Systems, a deciding factor in the award of the Kariba contract, is available to Electricity Undertakings all over the world. Ask to see the Company's specialist engineers at the planning stage.

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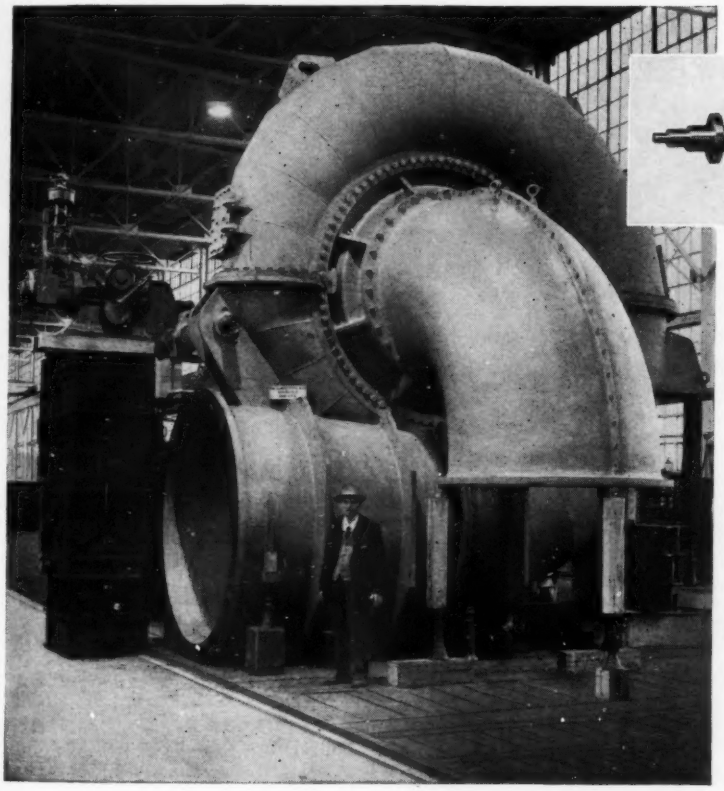

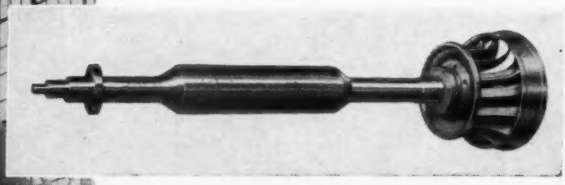

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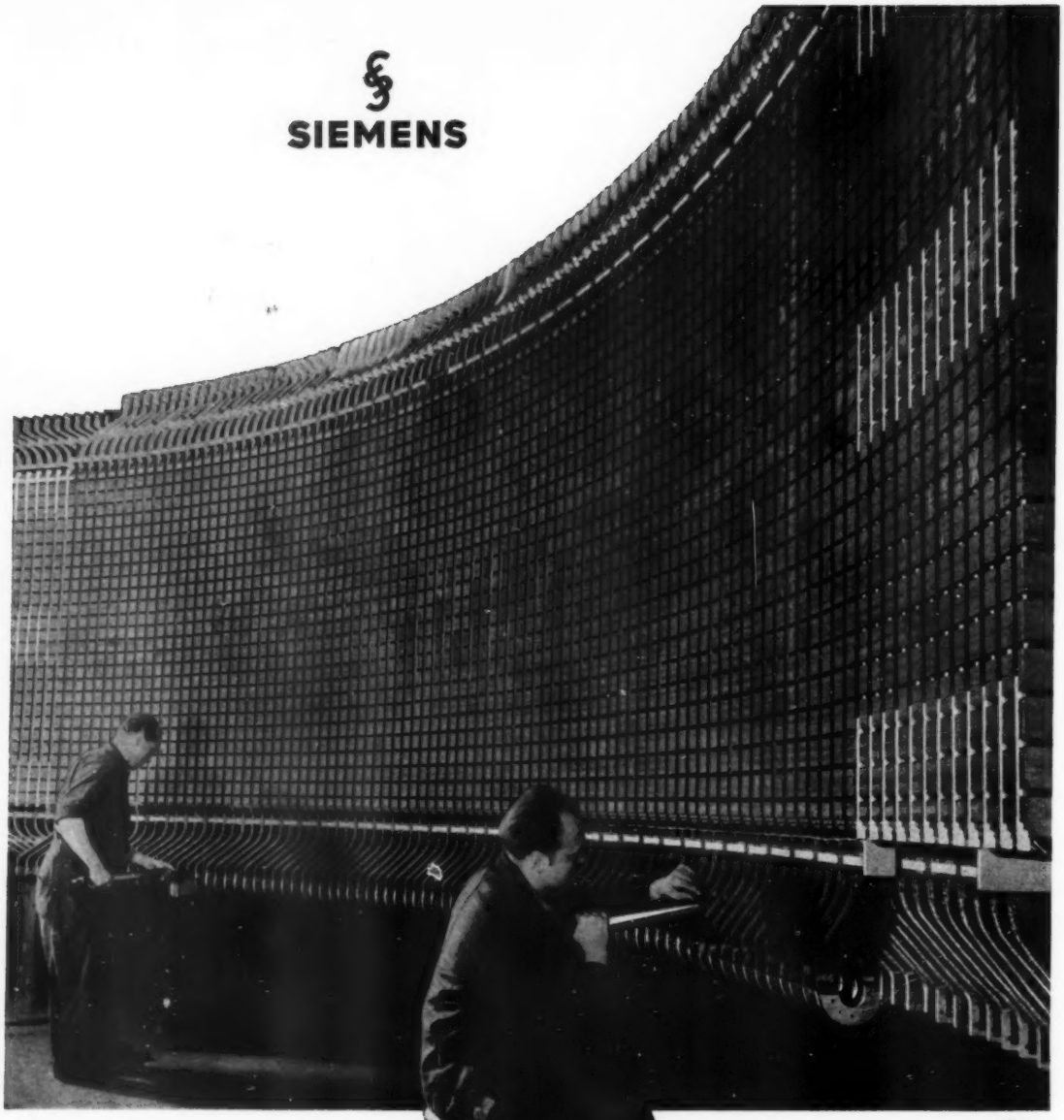

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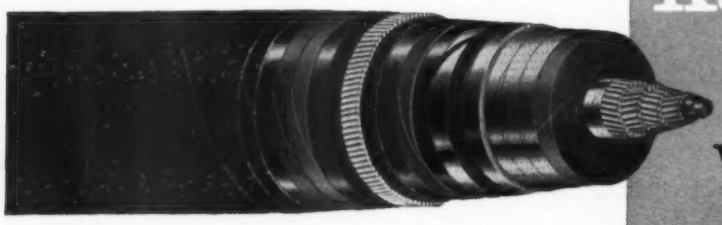

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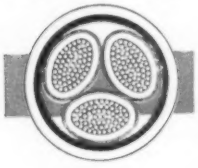

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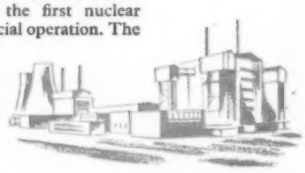

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WATER POWER

A technical journal devoted to the study of all aspects of hydro-electric development

Dynamic Engineering

ORGANISATION of time and equipment, whether it be material or human, is a subject which most busy engineers must find interesting. As techniques improve, the pressure of an average working day increases because an energetic man finds that the only limit to his capacity for work is fatigue; and fatigue can be greatly reduced if the maximum use is made of the modern facilities available.

We had occasion recently to accompany a busy European public-works engineer during a period of about 12 hours which, for him, constitutes a normal quiet day. The engineer in question is responsible for all state civil-engineering projects and maintenance over an area of about 500 sq. km.; among his responsibilities are roads, reservoirs, water-supply systems and two remarkable hydro-electric schemes, both of which are at present under construction. It is evident that no single individual can undertake unlimited work without delegating responsibility to competent aides, but it is only because this engineer is suitably equipped that he can give adequate supervision to his many undertakings.

Within his organisation there is a large pool of vehicles, many of them being trucks and vans for use on road construction and maintenance. There is also a section comprising 12 cars or jeeps, each of which is fitted with a VHF Siemens transmitter/receiver unit; the channel system employed enables any vehicle to call one or more of the others, or to speak to a central exchange through which the communication can be passed to any other telephone in the world. Similarly, the central exchange is in permanent contact with the mobile units, thus enabling the chief engineer to make maximum use of limited qualified personnel without recalling individuals to his office, an expensive and tiresome procedure where large distances are involved.

In this particular case, the methods employed for the supervision of two major hydro-electric schemes some 30 km. apart were most impressive. Problems requiring an immediate decision at either site were dealt with in a matter of minutes, and sometimes seconds, during road journeys round the area. On one occasion a series of appointments was made (involving the presence on site of fourteen consulting engineers and contractors' representatives) while travelling at a speed of 60 k.p.h. over mountainous roads. Such complicated telephoning while driving is made much easier by a two-way loudspeaker system enabling the executives to keep both hands on the wheel. Although V.H.F. "walky-talky" is common enough on hydro-electric sites it is unlikely that it has ever been used to the same extent as it is in this case, where local problems of economy and manpower make it essential for the chief engineer to hear, if not to see, all that occurs. In sparsely populated territory, such a system has many other uses, such as requests for medical help and advice of all kinds.

In this connection, we were also interested to see an example of good public relations between a chief public-works engineer and the population. Dams, power stations, and the like are notoriously unpopular with those who live nearby, although many objections are withdrawn when a project has become reality and the compensations are more fully realised. One of the schemes mentioned has involved considerable changes in the scenery near a village, but any adverse effects have been alleviated by extensive landscape gardening over an area of several acres; when the plants, trees and shrubs have matured, the added attraction to tourists will help to increase local prosperity. One of the many everyday problems dealt with during our 12-hour visit was the question of which shrub would best adorn a small plot near a subsidiary run-of-river power station. Compulsory acquisition of land is often an unfortunate but very necessary byproduct of development, but good public relations on a simple and unsophisticated plane can go far in helping to explain the need, or mollify the unwilling vendor. Similarly, a scheme may well change the whole social and economic pattern of the area in which it is constructed, and although each contractor may make adequate provision for his labour, the impact of a semi-itinerant force on local inhabitants must often cause friction of one kind or another; here

also the chief engineer becomes an important link, providing that he can make the time to listen, understand and advise. Despite the fact that public relations has become a profession unto itself, public-works and civil engineers are often better able to combine the duties of a liaison officer with their normal function; the term "public relations" generally implies indirect publicity and the propagation of information rather than the actual process of direct contact with the local population. The moral would seem to be that however busy an engineer may become, some part of his day should be devoted to the social and economic wellbeing of those directly, and sometimes brutally, affected by the development for which he is responsible.

There are many other items of equipment, other than the radio-telephone, that can be used or adapted on a large-scale hydro-electric development. Photographic gear of every description is essential and, in the vehicle which transported us during the 12 eventful hours, cameras, tripods, range finders, light meters and the like were stowed away on one side of the boot compartment, the other side being reserved for such items as a slide rule, office equipment, spare clothing and first-aid supplies. The automobile thus becomes as essential a companion to the engineer in the field as it is to any busy general practitioner, but how much time and fatigue would be saved if all that is now done by road were done by air. In very difficult country the helicopter will doubtless be the everyday transport of the future and it would not be too fanciful to suppose that the engineer we accompanied might efficiently supervise twice the area in half the time if he were airborne.

Snowy Mountains Annual Report

THE Snowy Mountains Hydro-Electric Authority have recently published their ninth annual report for the year ending June 30, 1958. During the year the Guthega project was operated as a run-of-the-river station pending the construction of the Kosciusko reservoir and produced the maximum demand of 67,400 kW on four occasions. Routine inspection and overhaul were carried out on machine no. 2, surge chamber, headrace tunnel and penstocks, and all were found to be in a satisfactory state.

Construction activity was concentrated on the Upper Tumut development, and the main and saddle embankments of the Adaminaby dam required 4,356,400 cu. yd. of fill, thus completing the dam. Furthermore the turbo-generators nos. 1 and 2 of T1 power station would become operational in mid 1959—their operation was reported in WATER POWER in the August issue 1959, p. 285—and it was expected that the full capacity of 320,000 kW would be produced by the end of 1959. The water in the main reservoir, known as Lake Eucumbene was gradually increasing and the reservoir then held 161,000 acre-ft. of stored water. It was also agreed to introduce trout into the lake.

This report indicates that four miles of the Eucumbene-Tumut tunnel have been constructed. In the October 1959 issue of WATER POWER it was reported that 4½ miles were already completed. This tunnel is to be 14 miles long and 21 ft. in diameter, and upon studying the rock conditions it was found that full concrete lining is unnecessary and that only 30% of the length would be fully lined in some sections. Other sections will only have concrete inverts. During the year 60,000 cu. yd. of concrete were placed in linings and inverts.

Very good progress was made on the 290 ft. high Tumut Pond dam. All the concrete in the dam and in the auxiliary works was placed by the end of the year. This four-year contract was completed in just under two years. Of the ancillary work carried out during the year covered by this report, is the completion of the road bridge across the Tumut river near T1 power station. This allows access to the tailrace tunnel portal and the headrace dam for the T2 project. The road system was extended by a further 42 miles, while a further 100 miles of existing roads were improved or reconstructed. Subsequently loads of up to 120 tons comprising electrical and mechanical plant were moved successfully by road. Construction operations have also begun on the Tantangara dam scheme whereby tunnelling operations from the upstream portal are under way.

In a comprehensive study of the most economic diversion of the Snowy River to the Murray, it was found that considerable improvement could be obtained if the Snowy-Murray diversion tunnel were located 700 ft. higher than originally proposed, and this was recommended to the government. Other studies were also conducted which include the testing of rock conditions, river flows of the Tumut and Upper Murray rivers under flood conditions and the rate of snow-melt. Cloud seeding experiments were continued and also engineering surveys on a wide variety of work were conducted.

Plans for the Lule River

SWEDEN is having to press steadily northwards in harnessing its water power, and during the next 10 years upwards of two thirds of the country's new development will be on the Lule river. This river, on which Harsprånget station already stands, contains the most favourable remaining sites for large and cheap stations.

On the Lule river the largest station in Sweden is now under construction at Messaure, an outstanding feature of which will be an earth and rockfill dam 100 m. high, 1,800 m. long, and containing 10·5 million cu.m. of fill. The station, of 240 MW capacity, will commence operation in 1962 at reduced head, and the scheme will be completed in 1963.

Another station under construction on the Lule is at Porsi, to have a capacity of 150 MW, a 120 MW station has just been commenced at Laxede, and a 250 MW project at Letsi is scheduled to be commenced next year.

Second Dam for Hells Canyon

TWO horseshoe-section tunnels, 42 ft. in diameter, are being driven through a rock promontory along the Idaho-Oregon border as part of a hydro-electric scheme which will have a capacity of 200 MW. They form what is one of the major features of the Oxbow station which is being constructed by the Morrison-Knudsen Company, Inc., for the Idaho Power Company.

This plant is the second of three schemes designed to harness the potential power of the Snake river over

a 100-mile stretch of Hells Canyon, and is about 13 miles downstream of the Brownlee dam, which has a potential of 360 MW and which was completed last year. The final scheme on this stretch of the river will be the Hells Canyon dam which will be designed for a capacity of 275 MW, so that the ultimate capacity of all three sites will be 1,000 MW.

The two Oxbow tunnels will be 800 ft. long and are being driven by jumbos each carrying 20 rock drills, the entire rig being mounted on a set of crawler tracks to assure mobility. The 205 ft. high impounding dam is expected to be completed next year and will require a total of about 1·5 million cu. yd. of fill, the concrete spillway portion already being virtually completed.

Loch Awe Scheme

GEORGE WIMPEY & CO. LTD. have been entrusted with two contracts in connection with the North of Scotland Hydro-Electric Board's Loch Awe pumped-storage scheme. The larger contract is for the main works of the Inverawe section of this scheme and includes a concrete-lined tunnel 23 ft. 6 in. in diameter and about three miles in length, a surge shaft 80 ft. in diameter and about 110 ft. deep, and the excavation of the power station site. The tunnel will be mainly in granite and is to run from a barrage at the mouth of Loch Awe through Ben Cruachan to a power station about half a mile upstream from where the River Awe enters Loch Etive.

The same firm have also been awarded the contract for the construction of a road, three miles long, from the main Dalmally-Oban road to give access to the site of the dam which will be constructed on Ben Cruachan as part of the pumped-storage section.

A. M. Carmichael Ltd., Edinburgh, have also been given a contract for the construction of a road, three miles long, from the Taynuilt-Kilchrennan road to Loch Nant, where a dam will be built to provide the storage reservoir of the Nant section.

The total value of these contracts will be just under £2 million. The estimated cost of the whole scheme, when published, was £24,500,000. The civil engineering consultants for the Cruachan and Inverawe sections of the scheme are Messrs. James Williamson & Partners, Glasgow; for the Nant section Messrs. Babtie, Shaw & Morton, Glasgow.

Power Situation in Europe

UNDER the adverse economic climate of Europe in 1957/58 and in spite of a slackening in use of primary energy, electric power consumption continued to advance by 7% to replace directly used primary fuels. This is stated in a study on the electric power situation in Europe in 1957, recently issued by the Secretariat of the United Nations Economic Commission for Europe (ECE). The report has a twofold purpose, namely, to provide a survey of the main facts and present tendencies affecting the industry; and to draw the attention of the ECE Committee on Electric Power to salient questions which may require consideration. The present document points to a number of basic features of the situation in Europe, including the importance of certain industries, such as metal manufacturing and chemicals, and of household consumers, in maintaining a high growth of consumption even in the face of recession; the progress planned in interconnecting various European countries so that all except three will, in a few years, be able to import and export electric power across their frontiers; the high rates of construction foreseen in the immediate future for hydro, thermal and nuclear power plants and extra high voltage transmission lines; a notable technical advance in automatic power-frequency regulation that now allows groups of adjoining countries to maintain complete flexibility in parallel working and easy interchange of electric energy at short notice. The report may be obtained from the Sales Section, European Office of the United Nations, Geneva, Switzerland, or may be ordered through Sales Agents for United Nations Publications. Price: $0·50 or 3s. 6d. sterling.

Hydro Resources Favoured in New Zealand

FOR the next 15 to 20 years the increasing demand for electricity in New Zealand can probably be most economically met by continuing to develop hydro resources. This opinion was expressed by the secretary of the Department of Scientific and Industrial Research, Dr. W. M. Hamilton, when addressing the Wellington branch of the New Zealand institute of Public Administration. Dr. Hamilton added that as the most favourable sites would probably be developed first, the tendency would be for costs per installed kW to rise as development progressed. This phase might be delayed substantially if the North Island could draw on South Island resources by means of a Cook Strait power cable.

At present nuclear power plants were too expensive to consider, and even should the cost of nuclear power in the future fall slightly below that of hydro power, there may still be good reasons for continuing to develop the country's hydro resources to the full.

Hydro stations made smaller demands on overseas funds for their erection and none for fuel fabrication or processing. This would also leave New Zealand independent of overseas supplies in time of war. He considered that geothermal power could make a useful contribution, but that it promised no solution to North Island power problems.

Hydro Development in Ceylon

CEYLON'S Minister of Transport and Power, Mr. Maitripala Senanayake, proposes to reorganise the hydro-electric supply industry of the island to bring it completely into line with the systems in operation in more advanced countries like Great Britain and India. To this end a statutory public corporation known as the Ceylon Electricity Board is to be set up to operate the industry in its entirety, with the temporary exemption of the Gal Oya development system.

The Minister is now seeking Cabinet approval to go ahead with this scheme. He has pointed out that the present structure of the electricity supply industry is unsatisfactory in relation to existing requirements and a radical change is necessary in order to deal adequately with its expansion in the national interest. Objection to the Minister's scheme has been raised by local bodies, on the ground that the scheme will result in their losing a large slice of their revenue. But the Minister gave several reasons why there should be a single authority for electricity. He stated that

with several independent local bodies the electricity distribution system could not be planned or operated in the most economical manner. Also he put forward the conclusions reached by Mr. J. C. Park, the British electricity expert, in his recent report after a survey of the current position of the industry in Ceylon, that it had now reached the size and stage of development when generation, transmission and distribution should all be integrated under a single management.

British Columbia's Expansion

THE first two 82,000 h.p. generating units at B.C. Electric's Bridge River No. 2 hydro-electric project are now in service. The 164,000 additional horsepower will consequently be able to meet peak demands this winter and bring up the B.C.E.'s total generating capacity to nearly 1,300,000 h.p.

Each of these new generating units produces enough power to supply all the homes in a city the size of Victoria. This project, which has been under construction since 1956, will cost $56 million and will be completed next year, when a final pair of generators are brought into service.

Mission dam is an earthfill structure stretching 1,200 ft. across the Bridge River valley and will be completed late next spring, when the flooding of the 750,000 acre-ft. reservoir will begin. This second phase of the project will fully harness the hydro-electric potential of the Bridge River area, bringing it up to 680,000 h.p. The capacity of the Bridge No. 1 plant is 248,000 h.p. and satellite plants at Seton and Lajoie add a further 88,500 h.p. to the system. The increased head of water created by the new dam will add another 25,000 h.p. to the generating capacity of the No. 1 station.

Delhi May Lose Power from Nangal

THE supply of electricity from Nangal to New Delhi may have to be cut following the recent mishap at Bhakra dam, upsetting the entire schedule for power production. Delhi gets 20 MW from Nangal. This fear was expressed at a meeting of the Delhi Industrial Advisory Board. It was stated that the supply of 40 MW of additional power scheduled to begin between December 1960 and March 1961 would be delayed.

It was understood, however, that there was no cause for alarm or pessimism in regard to the recent mishap to the hoist chamber of the Bhakra dam and that all steps were being taken to meet the situation. Following the lowering of a 40-ton steel gate in the cable gallery at Bhakra dam, the work on the generator floor has been resumed and all material lying on the floor has been salvaged.

The chairman of the Delhi Electric Supply Undertaking stated that its total generating capacity at present was 64 MW; in addition, the Undertaking receives 20 MW from Nangal. Of this, the firm supply capacity is 72 MW, as against the existing demand for 74 MW.

The work of erecting two used generating sets purchased from the Defence Ministry with a total capacity of 2,000 kW is in progress. A diesel set of 2,000 kW is being set up at Lahori Gate, while another diesel station with a capacity of 8,000 kW will be put up at Kilori.

The Undertaking has also decided to install a 30,000 kW steam generating plant. The Undertaking has under consideration plans for augmenting the generating capacity by 100,000 kW to 120,000 kW during the third Five-Year Plan.

Increasing Electric Power to Norwegians

DUE to Norway's increasing hydro-electric schemes it is now noted that more than 98% of the population have regular supplies of electric light and power. Due to various Government subsidies over the last 20 years, about 651,000 Norwegians have been supplied with power, and 37,000 people have had improved supplies. These subsidies, amounting to 335 million kroner (£16,750,000), have been used to this end, and a further 29 million kroner (£1,450,000) have been recently granted by the parliament for more improvements. This is due to the fact that there are still 44,000 people who do not have access to electricity, most of whom live northwards of Tröndelag. An article on Norway appears elsewhere in this issue.

Priest Rapids in Operation

THE Priest Rapids contracts, worth some $30 million, were won in the second half of 1956 by English Electric Export & Trading Co. Ltd. in conjunction with an internationally known New York construction firm, the Merritt-Chapman and Scott Corporation. For these contracts, The English Electric Co. Ltd. are supplying ten 114,000 h.p. water turbines and generators and five 183,000 kVA transformers to raise the generating voltage of 13,800 to 230,000 V for transmission to the various consuming centres.

The Priest Rapids project is one of a chain of stations being built to harness the power of the Columbia River system for distribution in the Pacific North-West. The site of the project is downstream from Grand Coulee, Chief Joseph and Rock Island dams, but upstream of the McNary dam, and above the confluence of the Snake and Columbia Rivers.

The turbines are of the vertical Kaplan type with a nominal rating of 114,000 h.p. at a 78 ft. head but are capable of producing 131,000 h.p. at 84 ft. head. The vertical umbrella-type generators each have a nominal output of 83,000 kVA at a speed of 85·7 r.p.m. but are capable of producing 95,500 kVA, the power factor being 0·95 in both cases.

New Zealand North Island Scheme

THE New Zealand Government has decided to proceed with the construction of the Matahina power station on the Rangitaiki River in the Bay of Plenty, North Island. The first power is expected by the winter of 1965. The costs, including connection with the North Island 220 kV transmission system, and housing and hostel for staff, are estimated at £8 million. The station will generate 300 million kWh a year from two machines with an aggregate capacity of 70 MW. Tenders for the construction of the diversion tunnel, which will be 25 ft. in diameter and 8,000 ft. long, will be called before the end of this year. An earth dam of approximately 3 million cu. yards, with a crest length of 1,300 ft. and a height of 197 ft., would retain the water at a maximum elevation of 250 ft.

Fig. 1. Aerial view showing intake and penstocks under construction

Três Marias Dam

This key project for a major power and reclamation development in Brazil is a joint undertaking by Comissão do Vale do São Francisco and Centrais Elétricas de Minas Gerais S.A.

PART ONE

A VISITOR travelling across the most remote areas of Brazil will observe that a great construction movement is in process from the coastal fringe towards the west. The country occupies the world's third position with respect to usable land, but, so far, more than half of its vast territory has been practically untouched.

The observed demographic expansion of 2·3% cumulative per annum, applied to the present 63 million population, will lead to a 100 million in 1980. This considerable growth in population has forced the need to integrate the virgin regions with the rapidly expanding economy, and the present Federal Government has prepared a long-range plan for the hinterland and launched an extensive programme of public works in the Brazilian middle-east.

Among the boldest projects are:—the construction of a 4,700 km. highway—Transbrasiliana (BR-14)—connecting the extreme south to the mouth of the Amazon river, through the plateaus of Paraná, São Paulo and Goiás and traversing 500 km. of the Amazonian jungle; the construction of Brasilia, the new capital (seated on the common divide of the large basins of the Amazonas, Paraná and São Francisco rivers), which alone required the construction of several highways and railroads; many large hydro-electric projects; and a large reclamation programme in the São Francisco valley, including the Três Marias multi-purpose dam (Fig. 2).

The São Francisco Reclamation Programme, which aims to raise the economic standards of this valley, covers a surface of 620,000 sq. km. (about six times the size of Tennessee river watershed in the United States) and comprises lands in five states. Its main purposes include the regulation of the river discharges and the development of hydro-power potentialities, improvement of the navigation and other means of transportation, introduction and development of large-scale irrigation, flood control, the fostering of industry and improvement of health and immigration conditions.

To organise and carry out specifically the general plan for the São Francisco valley, encompassing all

the tasks mentioned above, the Brazilian Congress in September 1946 gave the requisite authority to the newly created São Francisco Valley Commission. This article will describe the Três Marias dam, the first large undertaking of the Valley Commission and its key project.

The São Francisco Valley and the Três Marias Project

The São Francisco River has its source in the central mountain range in the State of Minas Gerais, at an altitude of 1,400 m., and runs practically parallel to the coast in a direction N.N.E. for 2,200 km.; it then abruptly changes its course to N.E.-E. for 543 km. before flowing into the ocean. The present navigation extends over a 1,300 km. stretch of its middle course, from Pirapora to Joazeiro, and over the entire lower portion (300 km.) which begins just downstream of the Paulo Afonso Falls. Great monthly variation of discharges (120 to 2,000 cu. m. per sec. for the driest year in Pirapora) restricts river transportation to small barges during the drought periods.

Climate and hydrology vary gradually as the river flows northward. In the Upper Valley, the climate is mild and dry with an average annual temperature of 18° C. and 1,400 mm. of rainfall. Along the Middle Valley the temperature increases very rapidly to attain 27° C. and the rainfall is less, amounting to only 400 mm. in the northern bend of the river (near the town of Cabrobó). The evaporation is extremely high in this section, with most of the rivers having only intermittent flows. The climate is therefore quite arid and the vegetation in this region is of the semi-desert type.

A complete pedological survey covering the whole valley, concluded in 1957, nevertheless indicated that even in the most arid zones a rational agriculture could be developed provided that water was furnished permanently; these conclusions have been confirmed by an experimental station in Petrolandia and the irrigated fields near Cabrobó, both situated in the driest region of the valley.

The Lower Valley, in spite of having a high average temperature (25° C.), gets the benefit of periodic rains brought in by coastal winds, so that the rainfall increases gradually towards the coast where it reaches 1,200 mm. per annum.

The population now amounting to 4 million is mostly concentrated in the upper reaches of the valley, in the central industrialised area of the State of Minas Gerais (around Belo Horizonte) and on the lower São Francisco, near the estuary. The Middle Valley, 1,600 km. long, is barely inhabited, with an index of 2·9 inhabitants per sq. km. and their existence depends on cattle breeding. Agriculture, due to lack of transportation and irrigation, is mainly confined to the alluvial plains along the main river and its tributaries. The heavy floods that occur periodically, though they cover the banks with fertile alluvial deposits, destroy the plantations and compel the tenants to move.

The São Francisco Valley Commission, when elaborating the basic comprehensive plan and examining the priority of the projects to be constructed, took into consideration all the natural and social aspects disclosed by the most modern means of surveying and research. One of the first conclusions drawn from the original plan was the need to control the river discharges, which would also result in greater benefits

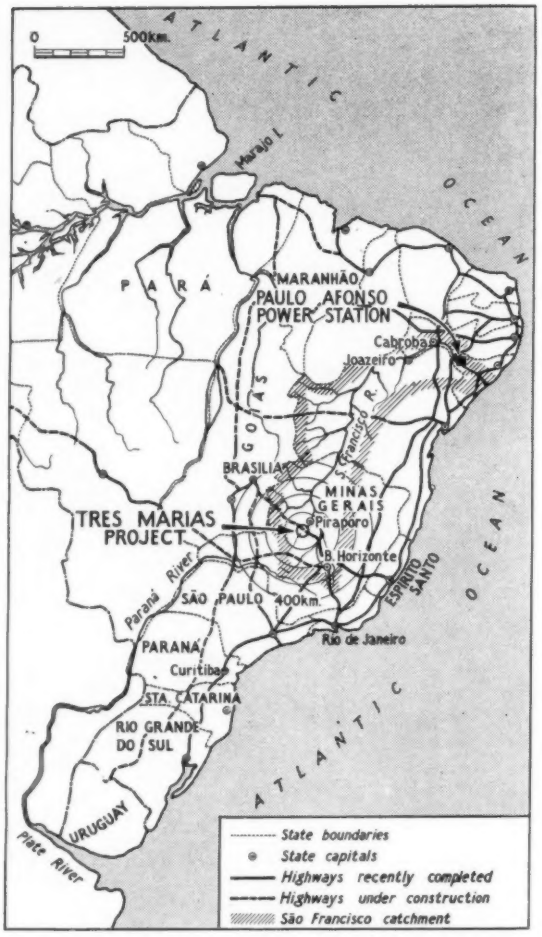
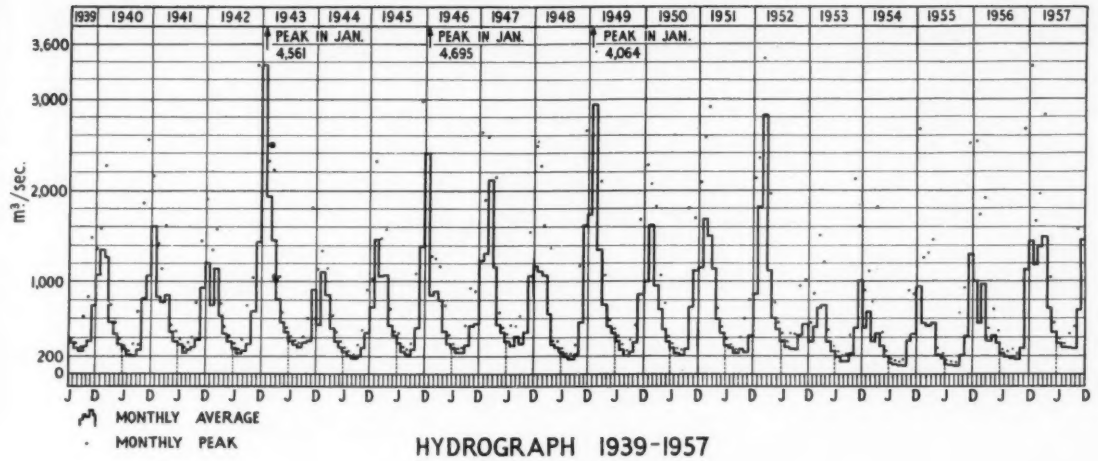


Fig. 2. Map of Brazil showing location of the Três Marias project

to the existing navigation, would relieve the populations and farms along the river from periodical flooding and lead to the development of a substantial amount of hydro power. As a first step towards harnessing the river, the location of a major dam was fixed upstream of Pirapora (the southern terminal of the navigation), which would enclose practically all the Upper Valley where the rainfall is substantially greater. Extensive land reconnaissances (confirmed lately by photogrammetry) indicated the rugged stretch of the valley between the junction of Paraopeba river and the town of Pirapora to be the only feasible region for a large storage dam. Comparative economic studies for three locations resulted in the selection of the site called Três Marias, located just below the mouth of the Borrachudo River, as being the most suitable for the purposes in view.

Três Marias dam, an earthfill structure which will be 2,700 m. long and about 70 m. high from the foundation, will impound some 120 billion cu. m. of water, creating a reservoir 150 km. long in the main river and 60 km. up the Paraopeba, its main tributary on the upper basin. The many benefits resulting from the erection of this dam will be to

(*a*) prevent flooding of some 335 sq. km. of the lower



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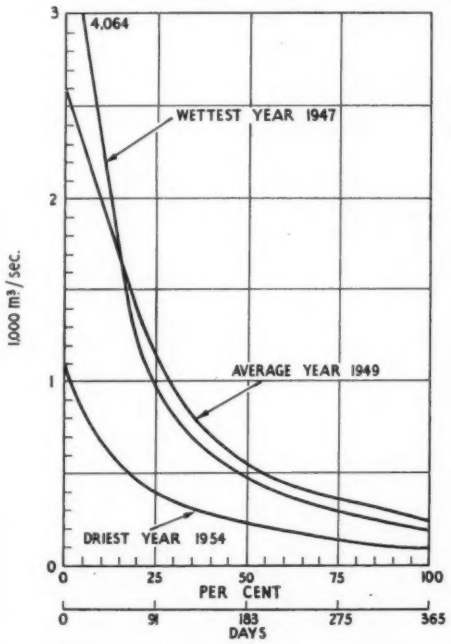
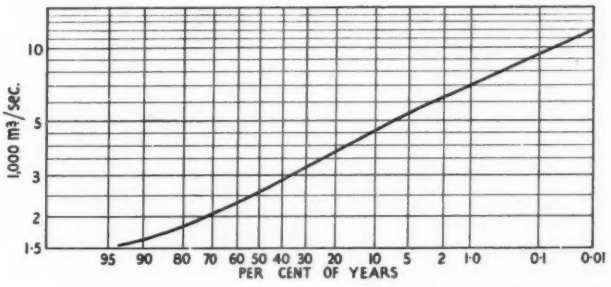
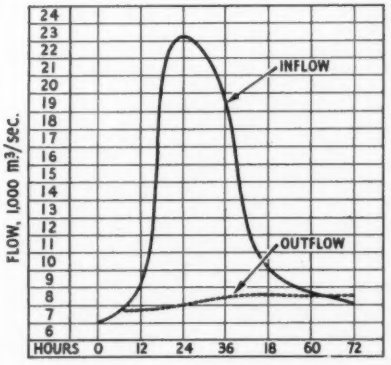


ANNUAL DURATION CURVES



FREQUENCY OF ANNUAL PEAK DISCHARGE



SPILLWAY DESIGN FLOOD

Fig. 3. Hydrological characteristics of the São Francisco River at Três Marias

fertile lands extending for 200 km. and secure partial flood control below this point.

(*b*) to ensure year-round navigation in the Middle Valley (1,300 km.), even in the driest years when a minimum 1·50 m. draft must be provided.

(*c*) enable the installation of a 520 MW power station at the toe of the dam with a production of about 2·7 billion kWh in an average year.

(*d*) firm up the power of all future downstream hydro-plants—in particular those of the Dois Irmãos and Pirapora projects, now under study, immediately below the Três Marias dam.

(*e*) permit some 400 MW extra firm capacity to be installed in the existing Paulo Afonso power plant (final firm capacity 1,000 MW).

The Dam Site

Hydrology. The hydrographic basin limited by Três Marias has a surface of 48,200 sq. km. at an altitude higher than 500 m. It is bordered on the south by the chain of the Canastra and Vertentes Serras, on the east by the Espinhaço Central N-S range and on the west by the Mata da Corda and Formoso Serras. The mean annual rainfall over the valley above the dam

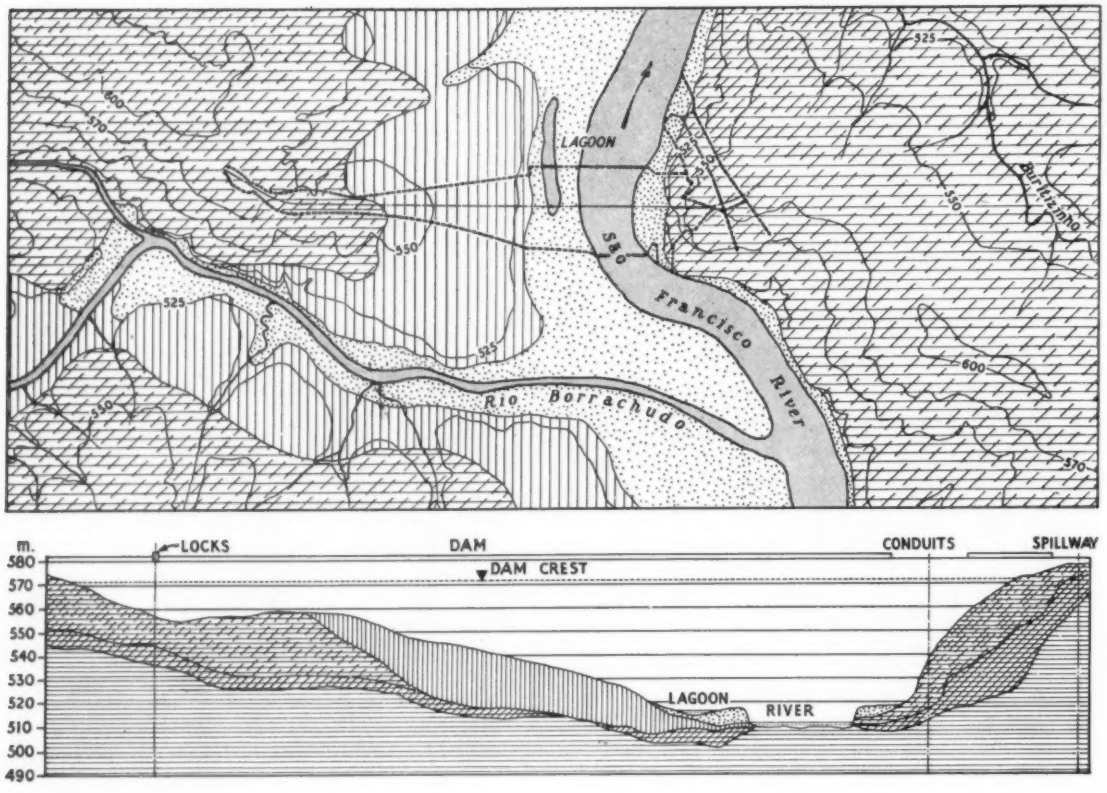


Fig. 4. Geological map and section of the Três Marias dam site

site is about 1,400 mm., with an average annual run-off of 13·5 l. per sec. per sq. km. or 420 mm. in depth, corresponding to about 30% of the precipitation.

It was found for a 19 year period that the mean annual discharge was 654 cu. m. per sec, the driest year occurring in 1954 with an average monthly discharge of 315 cu. m. per sec. During this year, which was exceptionally dry in all Brazilian south-eastern basins, the discharge during the low-water season (May-November) averaged 191 cu. m. per sec. The maximum estimated daily flow was 4,695 cu. m. per sec. which occurred on January 5, 1956. (Fig. 3).

Local Geology. The São Francisco river at the dam site is 300 m. wide, the depth in low flow varying from 0·5 m. to 2·0 m.; the elevation of the normal water surface is about 510 m. above mean sea level. The valley, due mostly to differential weathering, is markedly asymmetrical in profile; while the right bank is steep and shows frequent rock exposures, the left bank, rather flat, is formed over the first 300 m. by a flood plain 4·0 m. higher than normal river stage; this is followed by gentle slopes over the next 800 m. formed by terraces of Tertiary deposits up to El. 558, at which point the same residual formations found on the opposite bank begin (Fig. 4).

The dam foundation on the right bank and on the river channel lies entirely on Silurian formations of the Bambui Serié, consisting of metamorphic greenish-grey siltstones or sandstones and occasionally quartzites. Extensive drilling investigations on the right bank revealed an alternation of layers of sound and decomposed rock down to bed rock. The river bed, on the contrary, is completely lined by massive rock in the form of big slabs. On the left side, beyond the river edge, the bed rock is overlain by a band of alluvial sediments 300 m. wide, of Quaternary Age (Holocene) with an average thickness of 6 m. and consisting of dark silts.

The Tertiary formations that cover the major part of the left bank are alluvial fan deposits that have been built up by the Borrachudo River at the confluence of the São Francisco, at periods when both rivers were working over high levels (probably on the Pliocene). Sub-surface explorations by means of deep shafts revealed the existence of graded deposits varying from rounded boulders (30 cm. in diameter) to fine sand and clays. The remainder of the left bank consists of a thick mantle of residual clays and weathered stones topping the matrix rock (siltstone) which lies 20 to 30 m. deep.

From the structual point of view, the local rock, following the regional trend, has practically a horizontal bedding (the dip is about 2°) and shows two sets of vertical jointings, one of which is virtually normal to the axis of the dam. Mechanically speaking the local

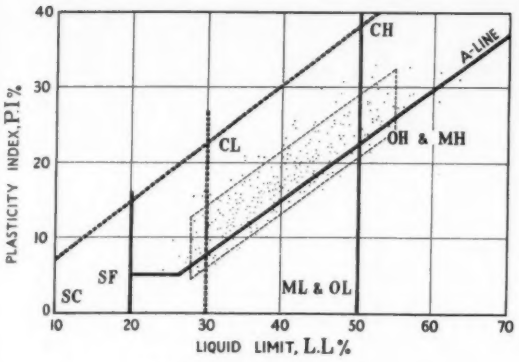


Fig. 5. Plasticity chart

siltstones and sandstones are quite massive, moderately hard and resistant to weathering. Their ability to act either as a foundation or as a coarse concrete aggregate was proved by intensive testing.

Field Investigations

Due to the relative remoteness of Três Marias and the great size of the undertaking, the field investigations at the dam site and in the general area of the project were carried out intensively and were followed by thorough construction planning. As it appeared from the reconnaissance of the site, the scheme that would best cope with the local conditions, making for a feasible and economical project, would call for an earthfill dam utilising the ample local clay deposits. The main facts leading to this conclusion were:—

(*a*) the absence of large rock outcrops on the foundation area;

(*b*) the great bulk of cement required for an all-concrete structure;

(*c*) the long-distance hauls for cement, lumber and other construction materials;

(*d*) lack of natural sand and suitable rock for concrete aggregate in the vicinity.

The field investigations were then conducted with three main objectives:—

(1) foundation investigations by sub-surface exploration;

(2) location and testing of concrete-making materials;

(3) sampling and testing of borrow materials to be used in the fill.

Sub-surface Explorations

Information about the conditions in the foundation area was obtained by 75 diamond-drill holes (mainly AX bits), 66 test pits, eight $1\frac{5}{8}$ in. diameter percussion holes and 18 2 in. diameter Shelby holes; the last three types were located chiefly on the silty clays of the left bank and furnished disturbed and undisturbed samples for the soil tests.

The main facts disclosed by the sub-surface explorations were:

(*a*) a great depth of overburden existed ranging from 7 m. to 30 m. (comprising soils and weathered rock) over the sound rock, except on the river bottom which consisted completely of sound rock;

(*b*) the local rock was free from faults and not subject to any major folding, but was jointed and required special means of protection against underseepage.

(*c*) the dark-grey silts in the flood plain area were in a saturated state below a depth of 2 m.

(*d*) a porous clay layer existed on the Tertiary formations.

(*e*) in a broad sense the soils in the foundation area were very impervious irrespective of their location and could very properly be grouped into three types:

(1) in the flood-plain area (left bank), the soils were highly plastic soft clays of the CH type, with lenses of gravel and fine sand interbedded. Triaxial compression tests (Q) performed on undisturbed samples, taken either from the upper drier or from the lower saturated strata, showed quite low resistance values.

(2) The Tertiary formations chiefly consisted of red silty clays of the CL type. It was found that the top 9 m. were in a very loose state with a low *in situ* density of 1·35 to 1·41 tons per cu. m. (porosity 47% to 50%) when compared with the lower layers. Compression tests (Q type), with measurements of volume reduction due to air compression, indicated that porous clay had resistance values of Ø=21° and C=0·2 kg. per sq. cm.; slow S tests on saturated specimens showed a 27° angle of internal friction. From consolidation-permeability tests it was estimated that about 86%, on average, of the total consolidation would occur almost immediately following the application of the load.

(3) The right abutment and the left end of the foundation area were covered by stiff residual clayey silts (ML and CL-ML) of pink, yellow and grey mottled colours. Resistance parameters

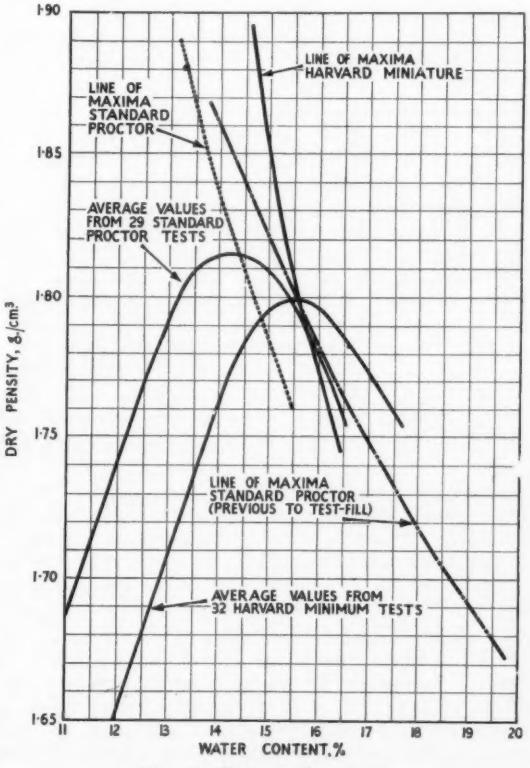
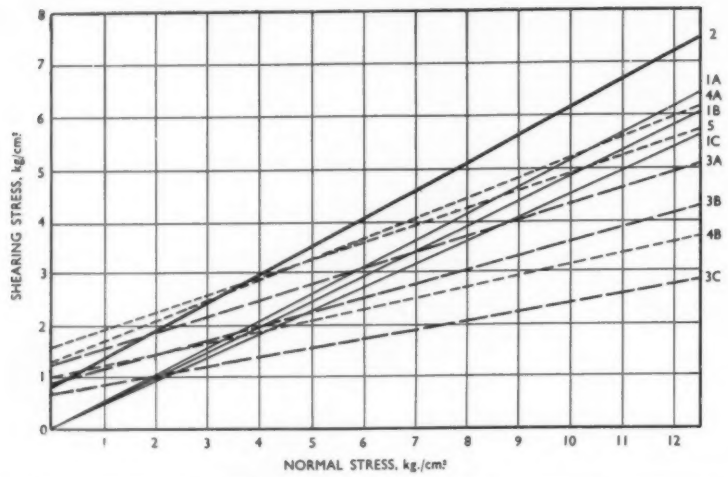

LINE OF MAXIMA HARVARD MINIATURE
LINE OF MAXIMA STANDARD PROCTOR
AVERAGE VALUES FROM 29 STANDARD PROCTOR TESTS
LINE OF MAXIMA STANDARD PROCTOR (PREVIOUS TO TEST-FILL)
AVERAGE VALUES FROM 32 HARVARD MINIMUM TESTS
DRY PENSITY, g/cm³
WATER CONTENT, %

Fig. 6. Compaction tests



ENVELOPE	TEST TYPE	AVERAGE CONDITIONS BEFORE TEST: DRY DENSITY	AVERAGE CONDITIONS BEFORE TEST: WATER CONSTANT	TEST STRENGTH PROPERTIES: ANGLE OF INTERNAL FRICTION	TEST STRENGTH PROPERTIES: COHESION	REMARKS
1A	S tests saturated			27°		Max. Strength, effective stress
1B	Qc tests with pore	1·79 g/cm³	14·6%	25·5°	0 kg./cm²	Ave. " " "
1C	pressure measurem.			24°		Min. " " "
2	S test, as compacted	1·83 "	15·4%	28°	0·8 "	Ave. " " "
3A		1·82 "	15·0%	17°	1·25 "	Max. " total stress
3B	Qc test saturated	1·81 "	15·1%	15°	0·9 "	Ave. " " "
3C		1·80 "	15·5%	9·5°	0·65 "	Min. " " "
4A		1·81 "	14·9%	21°	1·65 "	Max. " " "
	Q test, as compacted					
4B		1·78 "	14·9%	12°	1·0 "	Min. " " "
5	Qc test, as compacted	1·83 "	14·6%	18°	1·3 "	Ave. " " "

Fig. 7. Graph and table of compression tests

were:—Ø varying from 12° to 22° and C varying from 3·1 to 1·0 kg. per sq. cm., based on Q tests.

Soil Investigations in the Borrow Areas

Since the embankment would require about 14 million cu. m. of fill, extensive investigations had to be made in order to determine the quantities and types of soils available in the vicinity of the site.

As had become apparent since the beginning, very limited quantities of natural permeable materials existed in the region, most of the overburden consisting of highly impermeable silts or silty clays with small percentages of fine sand. This ready availability of large amounts of impermeable material immediately suggested the adoption of an earthfill dam of the homogeneous type, utilising also the fine sands from the Borrachudo River for the filters and the rock from both the excavation and the quarry for the riprap and drains.

The investigations for the borrow area were directed to the gently sloping areas on the left side of the São Francisco River, downstream of the dam and upstream of the junction of the Borrachudo River; the materials forming the low-lying level alluvial flood plains along the river, on the contrary, were proved to be unsuitable for incorporating in the fill.

The borrow areas were prospected in grid lines by means of auger borings and test pits, spaced about 75 m. and approximately 6 m. deep. Each test pit or auger hole was logged and the necessary samples taken as the hole was excavated. Sections along each line of holes, complemented by information given by index tests and visual classification, allow easy quantity computations to be made and a plan of routing each type of material from the borrow area to the dam site. About 18 million cu. m. of materials were prospected by means of about 600 auger holes and four test pits.

The index tests indicated that the fill materials could be classified as CL and CL-ML types and when plotted on Casagrande's plasticity chart would fall within the band parallel to "Line A, with liquid limits (LL) between 28% and 55% and plasticity index (PI) between 7% above and 1·5% below line A (Fig. 5). About 50% of the borrow-area soils have LL values above 40%. Such material was selected for the core of the dam and the less-plastic soils were chosen for the shell.

During the investigation phase, a test fill was executed utilising representative borrow-area materials and the equipment that would be employed in the construction of the dam. This test had the following objectives:—

(*a*) to determine the compaction properties of the available soils and the conditions and procedures for the fill placement;

(*b*) to obtain samples of field-compacted soils for laboratory tests to check the design assumptions;

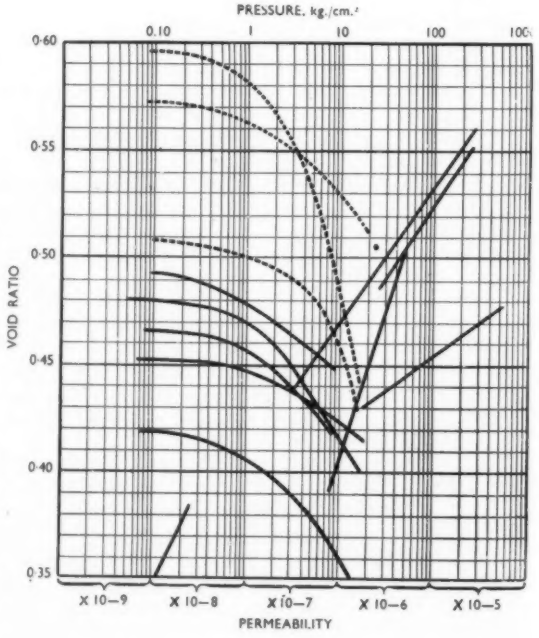


Fig. 8. Consolidation-permeability tests

(*c*) to train local personnel on compaction procedures and supervision.

Compaction characteristics for laboratory control were determined by standard Practor (ASTM D-698) procedure and with the Harvard miniature device; comparison of the results from both methods indicated small discrepancies, though within the specifications. Fig. 6 shows curves from representative borrow-area samples, the maximum dry density of the soils varying around 1·80 tons per cu. m. and the optimum moisture content ranging from 14% to 16·5%. The main borrow area presents soils with dry densities varying from 1·695 to 1·8 tons per cu. m. and an optimum moisture content ranging from 16% to 20%.

Triaxial shear tests were of three types: quick (*Q*), quick consolidated (*Qc*) and slow (*S*). In Fig. 7 Mohr stress envelopes are plotted for the various tests performed on the samples under conditions that will probably occur in the fill during construction and after the reservoir has been filled.

Combined consolidation-permeability tests revealed that the permeability coefficient varies from 10^{-6} to 10^{-9} and that at the river section the expected settlement in the lower layers will range from 3·7% to 5%. Fig. 8 shows the settlement and permeability curves under different pressures for samples taken from the test fill and also for specimens compacted at 98% of maximum Proctor dry density.

Investigations for Construction Materials

The general area of the dam site, within a radius of approximately 10 km., was fully reconnoitred in an endeavour to find an economically located quarry site, Geological studies had already reported the absence of rocks other than the fine-grained metamorphic siltstones and sandstones, always covered by a thick mantle of residual material. Owing to the deep overburden, solid rock outcrops, with the exception of isolated horizontal beds, are not usually found in this type of country; field reconnaissances were thus primarily limited to stream channels which had been cut through the overburden to the sound siltstones beds underneath. The best siltstone quarry was located 7 km. from the dam site on a high bluff, where the top soil had been denuded away and the rock outcropped on an easily workable face.

It was found cheaper to provide for primary and secondary crushing at the quarry, the downgraded products being hauled to the construction plant. Samples from this quarry were subjected to compression tests (ASTM c-170-41T), abrasion tests with the "Los Angeles machine" (ASTM C-131-44), sodium-sulphate soundness test (ASTM C-88), tenacity tests (with Tretton machine), in addition to practical crushing and concrete trials. These test results, complemented by field observations, indicated that the sound siltstones would be suitable for coarse aggregate and riprap, though they assumed a flat and elongated form after crushing. Crushing tests were also effected to determine the possibility of manufacturing sand from the local rock, but with negative results, as was expected, due to the fine-grained composition of the siltstones. The ultimate product of crushing resulted in extremely small particles with no gradation and containing an unacceptable percentage of dust. As known sand-making rocks were at least 150 km. from the dam site, the search for small-size aggregates was the object of considerable investigation based on the location of economic sources of river-transported sand. Finally the best choice was found to be the natural sand bars on the Rio das Velhas, near the town of Corinto, the railroad terminal for the Três Marias and connected to the project by an all-weather road. Haulage of sand is now made over a distance of 105 km., by truck.

(To be continued)

Allis-Chalmers to Manufacture in Italy

Allis-Chalmers Italiana, S.p.A., which is a subsidiary of the parent firm in Milwaukee, Wis., has acquired the factory and crawler tractor business of Vender, S.p.A., in Cusano, a suburb of Milan, Italy. Mr. Phillip Bauer, managing director of Allis-Chalmers International and a vice-president of the parent company, stated that the new plant in Italy would afford the organisation an opportunity to take advantage of the many market potentials in the European Common Market countries as well as the surrounding countries. Because of the prohibitive tariffs and transportation costs, he added, it has been difficult to compete in these countries with products made in the United States, although Allis-Chalmers would continue to ship products made in the United States to other parts of the world. The Italian plant employs approximately 500 people. It produces crawler tractors ranging from 60 to 325 h.p. The plant has more than five acres of manufacturing space plus substantial adjoining property. It has modern machine tools and well-trained personnel.

Allis-Chalmers International also operates plants at Essendine, England; Newcastle, Australia; and Mexico, D.F., Mexico; and has sales offices and representatives throughout the world. This new Italian branch forms yet another expansion.

Restocking Scottish Rivers

The North of Scotland Hydro-Electric Board have started restocking of Highland rivers from their salmon hatcheries at Invergarry and Contin. This work is the Board's contribution to Highland restocking and is designed to meet the arguments that hydro-electricity undertakings interfere with normal restocking by natural means.

The eggs came from two sources, the Glen Garry trap, where 850,000 eggs were obtained in autumn stripping, and the other is the Ceannacroc trap on the Moriston, which gave 140,000 ova. Another new trap has been introduced at the tailrace of the Lubreoch power station near the base of the new dam which enlarged Loch Lyon; from this source 250,000 eggs were taken, and of these 135,000 eggs were planted in the gravel of neighbouring burns. The balance went to Invergarry and will be returned to restock the Lyon.

An Explanatory Brochure on Steel. The British Constructional Steelwork Association, Artillery House, Westminster, have added B.S. 449 : 1959, price 3s. 6d., to their list of technical publications with the object of making more easy the task of the engineer engaged in designing steelwork structures. The brochure replaces B.S. 449 : 1948 which is now out of date.

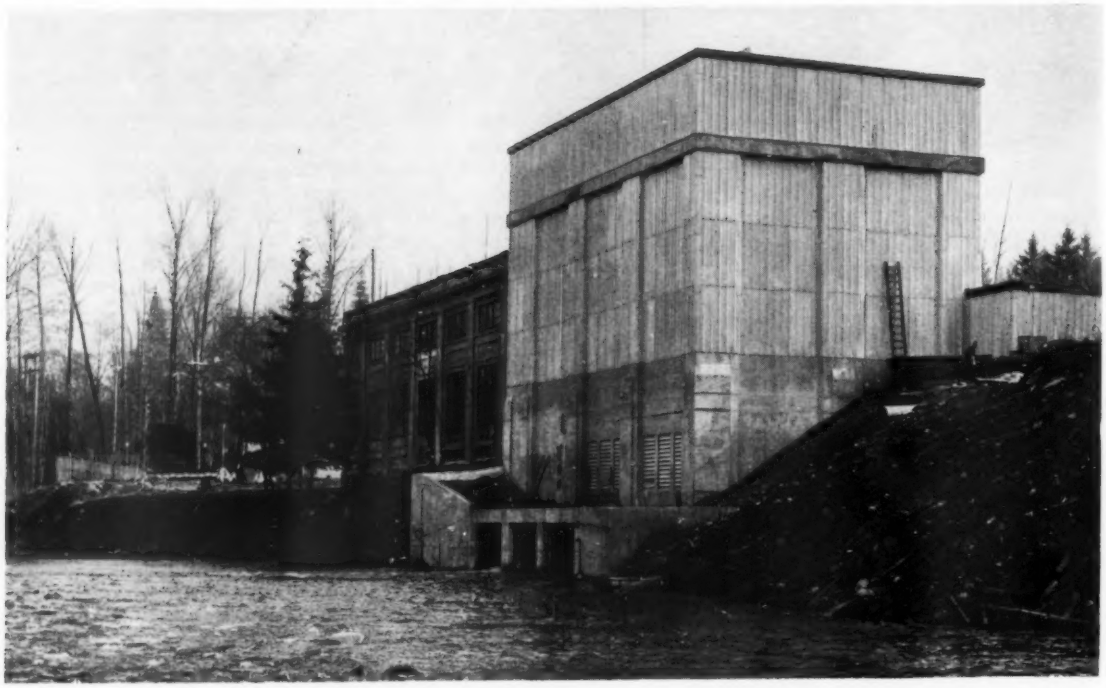


Fig. 9. Puntledge power station (in foreground) from downstream

Some Hydraulic Features of Puntledge Generating Plant

This article describes the pressure regulator and control system and gives a brief account of operating experience

By A. W. LASH and R. E. PASSMORE

PART TWO

THE pressure regulator is a device to control the rate of change of water flow in a penstock in a manner that will keep the pressure in the penstock within prescribed limits yet permit the flow through the turbine to vary widely and rapidly, thus maintaining close speed regulation with wide load changes.

If the turbine has to operate with rapid on-load changes as well as rapid off-load changes, a water-wasting regulator is necessary where negative pressures might damage the penstock. For off-load changes only, a water-saving pressure regulator can be used.

There are several different types of pressure regulator in use, one of these being the mushroom-disc type. Water or oil may be used as the operating medium for the servocylinder. The regulator may be "gate retarding," which, in case of failure of the regulator to open, mechanically retards closing of the gates by means of linkage connecting the regulator with the turbine gates; or the retarding action may be accomplished by throttling the discharge of oil from the turbine servomotors.

Operating Requirements for the Puntledge Pressure Regulator

For the Puntledge generating station the British Columbia Power Commission specified an oil- or water-operated pressure regulator with a preference for oil operation from the turbine-governor system. The plant would normally be operated on a uniform base or fixed load and under this condition the pressure regulator would be water saving, protecting the penstock for off-loads only. However, the regulator would be water wasting when it was required to operate the turbine on a variable load and provision for this feature was to be made.

The station was to be unattended, therefore provision for remote control was necessary.

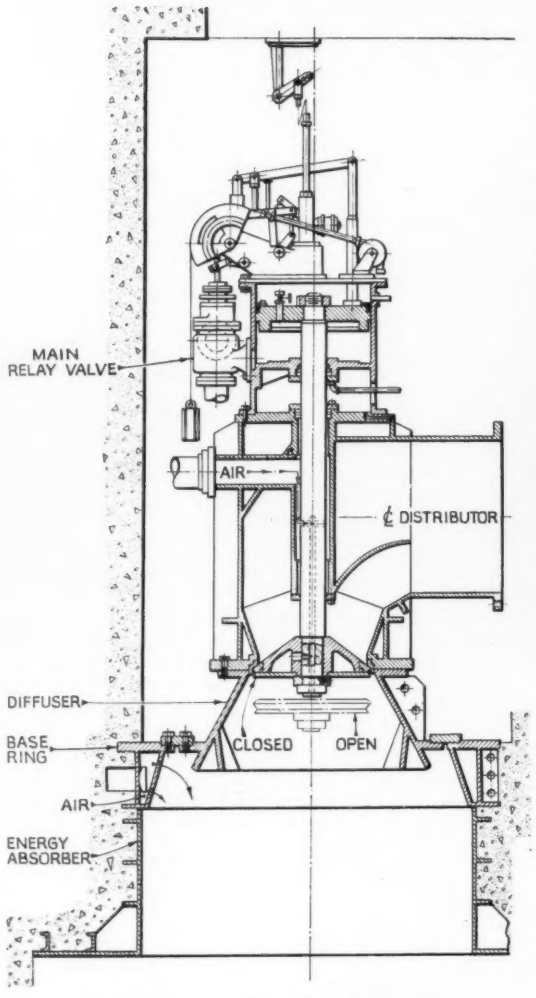


Fig. 10. Pressure-regulator assembly

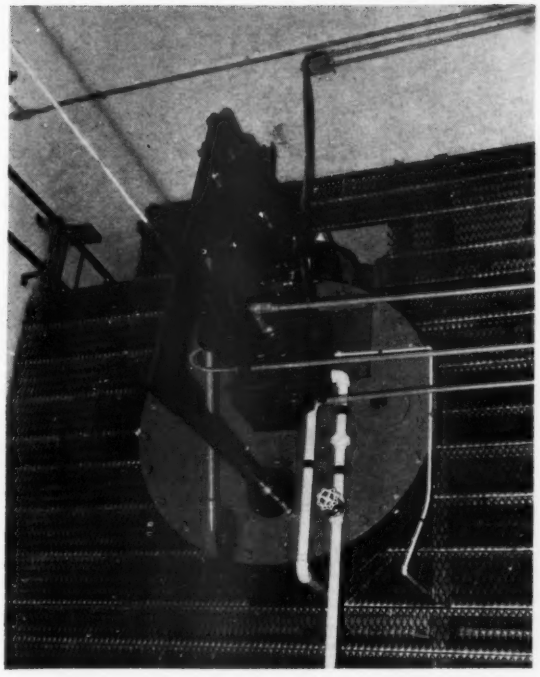


Fig. 11. Looking down on the pressure regulator

Design and Construction of the Pressure Regulator

The regulator shown in Fig. 10 is the mushroom-disc type "gate-retarding" and, for minimum servomotor size, oil was chosen as the operating medium. The regulator body is elbow-shaped and, together with the servocylinder, diffuser, and energy absorber, is of fabricated-steel construction. The only major parts made from castings are the mushroom valve, valve seat, and seal rings and the components constituting the main relay valve. The valve seat and seal rings are type 409 stainless steel.

Water discharging vertically downward past the valve disc enters the energy absorber through a cone-shaped diffuser or discharge pipe. This disperses the water into the energy absorber, and holes in the base ring surrounding the diffuser, together with holes around the periphery of the valve disc, permit the entrance of air into the water stream to aid energy dissipation.

The energy absorber is a simple cylindrical steel chamber closed at the bottom with its axis vertical. A rectangular, horizontal outlet welded to the cylindrical portion leads the water to the tailrace through a passage parallel to the leg of the turbine draft tube.

The servocylinder, mounted on the top of the regulator body is single-acting. It is a power cylinder on the closing stroke and a dashpot on the opening stroke. Thus only one cylinder volume of oil is used during one complete cycle of the regulator. The pressure-tank volume and oil-pump capacity are considerably reduced as compared with a regulator that requires a double-acting servocylinder. Because the turbine and pressure regulator act together, the pressure tank has to supply the oil for both servomotors at the one time. The total volume of oil used at Puntledge during one complete cycle, i.e., a closing and opening stroke of the turbine, is seven times what it would be if the turbine had no pressure regulator.

Another pressure regulator recently installed by Canadian Allis Chalmers Ltd. has by necessity a double-acting servomotor, resulting in a total oil capacity for one cycle of 15 times that for the turbine alone. It is thus seen that, when considering the use of an oil-operated pressure regulator, cognisance must be taken of the abnormally large pressure tank, pumping unit, and sump tank required. Although the Puntledge turbine requires only a 3 in. pipe size actuator, the pressure regulator has 8 in. oil lines.

The main relay or distributing valve is attached to the side of the servocylinder and the control mechanism is at the top with the mounting base doubling as the top cylinder cover.

The regulator is thus a completely self-contained unit, except that the throttling valve is mounted in one of the pipes between the turbine servomotors and actuator. When using a regulator of this type the sump tank should be installed below the elevation of the regulator cylinder and control mechanism

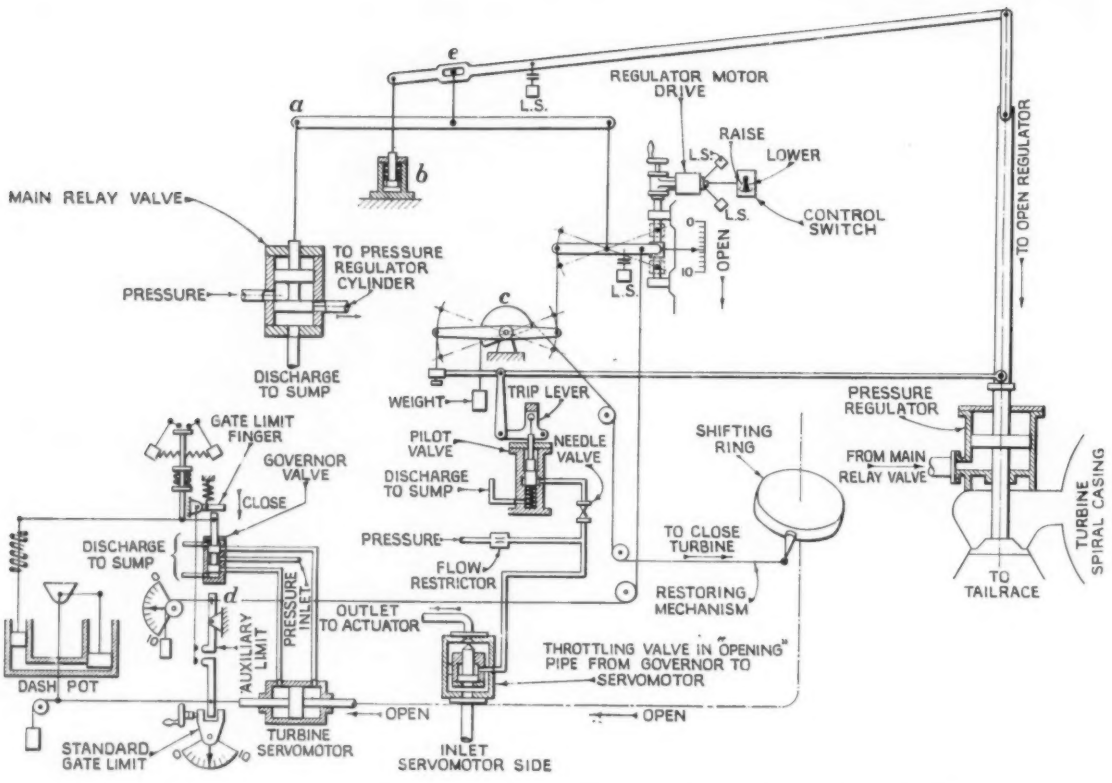


Fig. 12. Schematic arrangement of pressure regulator and governor control mechanism

to take care of the leakage of oil. This requisite presents a power-house construction problem because the elevation of the centre of the regulator inlet of this type of regulator is coincident with the elevation of the centre of the spiral casing. The regulator is thus deep inside the power-house and the sump tank would have to be in a deep pit. At Puntledge this is, unfortunately, not only impossible but undesirable because the oil pumps are mounted upon the sump tank. Therefore it was necessary to provide a small secondary sump tank with pumps and float switch to collect leakage oil and return it to the main tank.

The control assembly shown in Fig. 12 follows the same principle as that of a governor, with the gear motor likened to the speed adjustment of a governor. It includes the main relay valve, the pilot valve for the throttle valve, the lever assembly, and the gear motor for auxiliary gate limit or pressure-regulator opening-limit adjustment.

The main relay valve shown in Fig. 12 includes its servo and pilot valve. The control of the regulator is centred around the gear motor. By driving a lead-screw and lead-screw nut this motor sets the auxiliary gate limit on the actuator and imparts motion to the lever assembly. The position of the lead-screw nut determines the unit's discharge while the turbine load determines the turbine discharge. The pressure regulator takes up a relative position for the flow to remain constant at the value set by the lead-screw nut. For the pressure regulator position to be maintained, the main relay valve must be centred, and to do this, its pilot valve must also be centred. Thus point *a*, Fig. 12, must always return to the same position.

When the turbine is operating at the gate limit set by the lead-screw nut, the pressure regulator is closed, i.e., water saving, and the turbine cannot pick up additional load. Then if the lead-screw-nut gate-limit setting is reduced, the turbine gates will close in synchronism, the pressure regulator remaining closed.

If the nut is at a gate limit setting higher than that of the turbine gates, then the pressure regulator will open an amount equal to the difference, i.e., water wasting. In this position the turbine can rapidly pick up the equivalent additional load.

Should the turbine gates be closed by the standard gate limit on the actuator or prevented from opening by the same or other means, the gate-limit setting of the lead-screw nut will cause the pressure regulator to open an equivalent amount.

The purchaser specified that, as a water-saving pressure regulator, on load being dropped the gates were to close quickly and the pressure regulator was to open and maintain a constant discharge through the unit. This first sequence was to be followed by the pressure regulator closing slowly, at a rate which would limit the penstock pressure rise to the stated value. The Puntledge regulator meets the first requirement by the displacement and subsequent centring of the main relay valve. The slow closure sequence is achieved by manually switching the auxiliary gate limit to zero with the gear motor speed controlling the closing rate of the regulator. The water-saving cycle is thus seen to be not automatic; but rather

than being a disadvantage, this feature is an asset for the Puntledge Station. On a normal shutdown there is no necessity to close the regulator automatically. At other times a load rejection will be unintentional so keeping the regulator open allows the load to be assumed again quickly.

The regulator could have been designed to close automatically; but then, when it did, the majority of occasions would require it to be immediately reopened, the complete cycle taking approximately 15 min.

An important feature of this pressure regulator control, for which a patent is pending, is that it permits the turbine to operate *at any gate opening* with the regulator fully closed, i.e., water saving. Yet, when the gates close because of a load rejection, the regulator opens in such a way as to limit the pressure change to the value when closed from full gate.

Common designs of pressure regulators will do this only when the turbine is at full gate. Discharge of the regulator matches discharge of the turbine as the gates close from 10/10 to zero and the regulator opens from 0 to 10/10; but if the turbine is 8/10 and closes to 7/10 and the regulator opens from zero to 1/10, the change in flow of each will not be the same. This difference increases as the turbine runs at lower gate openings and the regulator remains closed, i.e., water saving. Therefore, from each gate or load position less than full load, the pressure change progressively increases. If the pressure regulator must control the pressure change within the specified value with the turbine at full load, the pressure regulator must be operated as water wasting on part loads.

This situation came to the attention of the author just recently. Another manufacturer's pressure regulator of the conventional pattern would not keep the pressure change within the limits necessary to protect the penstock when water saving and the turbine was at part loads. A surge pipe then had to be installed to supplement the action of the regulator.

A spring-mounted pivot shown at *b* in Fig. 12 at the extreme end of the main floating lever prevents the floating levers from overtravel during testing, should the pressure-regulator piston drop down when oil pressure is shut off and the controls are not properly adjusted.

The gear motor may be electrically operated either locally at the station or remotely. The gear may also be manually operated by the handwheel in the event of a power or line failure or when adjustments are made. When manually operated the electric motor is automatically declutched from the input shaft of the gear-reduction unit. Selecting a high gear-reduction ratio for the hand-wheel drive prevents excessive rate of closure when the handwheel is used.

The actuator, turbine, and pressure regulator together form a closed-loop system. The link between the pressure regulator and actuator, and the pressure regulator and turbine-gate operating ring is a weight-tensioned cable. The first part of this article mentioned that the turbine has a greater discharge potential than the pressure regulator, and both can discharge more water than that required for the maximum rated output. This was known when the equipment was designed, so provision was made on the turbine servomotors for the stroke to be limited after field tests had been taken. Similarly, the stroke of the pressure regulator servo-cylinder is limited by the travel of the lead-screw nut, which again was finally determined after tests.

Providing cable sectors *c* of different radii for the link between the gate-operating ring and pressure regulator enables the desired proportionate movement between the pressure regulator and gate-operating ring to be obtained by test. A slotted lever *d* in the link between the actuator auxiliary-gate limit mechanism and the pressure regulator lead-screw nut correctly proportions their respective movements. A slot at the fulcrum *e* of the main floating lever is a further aid in achieving precise proportioning.

Since the flow of water through the turbine and pressure regulator is not linear with their respective strokes, the shape of a cam compensates for these differences. Adjustment is made by changing the cam shape. No modification was done to the cam of the Puntledge regulator.

A control assembly similar to that on the Puntledge pressure regulator has recently been applied to a butterfly type of free discharge valve used both as a pressure regulator when the turbine is operating and as a discharge valve when the turbine is shut down. With the turbine gates closed, the control permits the valve to be opened to any point within its range independently of the turbine. The discharge capacity of the valve is greater than that of the turbine, yet the versatility of the control mechanism permits the turbine to operate from zero to full gate with full protection, the valve acting as a pressure regulator. And if the plant is required to pass more water than the turbine is capable of, the valve may be opened until the combined discharge of the valve and the turbine reaches that of the valve alone when it is fully open.

Associated Control Devices

1. The pressure-regulator limit switch is operated by the control linkage and normally prevents closure of the turbine-shutoff valve until the pressure regulator is fully closed, permitting closure of the butterfly valve only when there is no flow. However, on overspeed, a relay initiates closure of the butterfly valve regardless of the pressure-regulator position and simultaneously causes the pressure regulator to close.

2. A limit switch operated by the butterfly valve has one contact in the pressure-regulator opening circuit which prevents opening of the pressure regulator until the butterfly valve is fully open, thus, the butterfly valve can open only under a balanced-pressure condition and with zero flow.

3. The throttling valve and throttling-valve pressure switch protect the penstock should the pressure regulator fail to open. Many pressure regulators are equipped with a heavy mechanical connection between the turbine shifting ring and pressure regulator controls. This mechanical mechanism is designed to transmit full turbine servomotor thrust to the pressure regulator control. Since the throttling valve method of retarding the rate of gate closure in the event of regulator failure confines the turbine servomotor thrust within the governor oil pressure system, the mechanical connection between the turbine gate-operating ring and pressure regulator may be of much lighter construction. While the pressure regulator opens in synchronism with the turbine-gates

closing, the pilot-valve trip lever remains stationary. If, however, the pressure regulator fails to open, the trip lever moves and trips the pilot valve. This pilot valve then permits the pressure line normally holding the throttling valve open to drain through a needle valve, adjusted to prevent sudden closure of the throttling valve. Oil discharging from the turbine servomotors as the gates close can now close the throttling valve, restricting the flow of oil from the servomotors to the sump and thus slowing down the gate movement in the closing direction. Because the pressure line normally holding the throttling valve open is open to drain when the throttling valve trips, a flow-restricting valve is provided in the pressure line to limit the loss of oil from the pressure tank during the time that the throttling valve is held tripped. With the pilot valve closed, the flow is sufficient to overcome normal leakage past the throttling and pilot valves and still maintain system pressure in the line. The loss of the pressure that normally holds the throttling valve open, closes the throttling-valve pressure switch which, with the aid of a relay, initiates closing of the pressure regulator and trips the actuator shutdown solenoid. However, the unit is kept connected to the line, which prevents it from going to overspeed as a consequence of the slow gate closure. Before normal operation can be restored, the pilot valve and pilot-valve trip lever must be manually reset. An operator is thus forced to visit the station before the unit can be restarted, giving an opportunity for the cause of the pressure-regulator failure to be obtained. To date there is no record of the regulator failing and the throttle valve being closed.

4. Limit switches operated by the auxiliary gate-limit-adjusting gear motor open the motor circuit at each end of the lead-screw-nut travel to limit the movement of the valve disc automatically.

5. A limit switch operated by the lead-screw nut prevents closure of the unit start circuit until the pressure regulator is 70% open. This has been found in practice to be the optimum position for minimum penstock pressure changes when the unit is started.

6. Armature and field rheostats permit speed adjustment of the gear motor, and hence the opening and closing rate of the pressure regulator, when this is being done by the gear motor.

7. A pressure-regulator control switch is on the actuator for controlling the movement and direction of the lead-screw nut, which, in turn, opens or closes the pressure regulator. The amount the gates can open depends upon the amount the pressure regulator is open, so this control is referred to as the "auxiliary gate limit." A duplicate supervisory control is at the remote station.

A gate-limit indicator, additional to the normal gate-limit indicator, is mounted on the actuator. This indicator, together with a gate-limiting mechanism, is operated from the lead-screw nut. Thus the maximum permissible opening of the pressure regulator, and, in turn, the turbine gates, is visibly shown. A potentiometer provides indication at the remote-control point. Signal lights on the actuator and at the supervisory control indicate open (red) and closed (green) positions of the pressure regulator.

A criticism sometimes voiced regarding pressure regulators as a protection device is that the runners of turbines on which the pressure regulators are installed have a discharge capacity at runaway less than that at full-load synchronous speed. If a load rejection occurs and the pressure regulator fails to open, stalling of the wicket gates could permit the turbine to get to runaway speed with a resulting decrease in flow; such a condition would produce an objectionable pressure rise in the penstock. Each installation requires individual study to see if this pressure rise will exceed the permitted value.

The pressure regulator control at Puntledge allows the addition of a gate-operated limit switch in series with a speed switch to energise the control and open the pressure regulator should an excessive speed rise commence.

A study would determine the blocked gate at which an objectionable pressure rise begins to occur.

Operating Experience

The arrangement adopted at Puntledge generating station is an economical one both as to first cost and as to water use and is applicable where the plant can be operated strictly on base load, leaving other plants to do the governing. However, the plant can be readily connected to operation in a rapidly varying load by using the pressure regulator as a synchronous bypass. Wastage of water would, however, occur when operating in this manner.

The relative economy in plant cost arises from elimination of a very large surge tank which would otherwise be required and from reduced penstock thickness due to reduction of pressure surges.

While under normal operating conditions a small amount of water is wasted during start-up and shut-down operations, the amount is insignificant since the plant under all normal conditions is operating steadily with the pressure regulator closed.

The Kingsbury type of generator thrust bearing calls for a quick turbine-gate opening time in order to "break away" from the stationary position. Due to slight time lag and nonconcurrence of turbine gate and relief-valve discharge characteristics, this high speed can result in marked pressure variations. Although at Puntledge a completely satisfactory compromise solution of this difficulty was found, it would seem desirable in such a plant to consider use of a different type of generator thrust bearing which does not have any limitations as to speed of break–away.

The plant has now been operating for three years. No difficulties have been experienced in regard to the mechanical and electrical features of the automatic-control and supervisory equipment, nor on the operation of the relief valve.

Acknowledgments

This paper is presented by kind permission of the British Columbia Power Commission and Canadian Allis-Chalmers Ltd. The writers wish to acknowledge the assistance of Messrs. N. H. Latimer and J. W. Milligan, Engineers of the British Columbia Power Commission, who were engaged on the testing of the equipment and on the original computations on which it is based.

Fluid Level Controls. Elcontrol Limited, Hitchin, Herts., have issued a data sheet giving details of electronic fluid-level controls operated by means of probes making contact with the liquid at the levels desired.

Uplift in Gravity Dams

In this series the author reviews the various conceptions of the phenomenon of uplift and presents his own analysis supported by experimental evidence. This article discusses uplift pressure as a loading and considers the nature and value of the reduction coefficient

By SERGE LELIAVSKY, Ph.D., M.I.C.E., M.Am.Soc.C.E.

PART THREE

IT remains to show how to calculate the corresponding set of unit stresses so as to ascertain that the dam is not subject to tension. In this connection, attention is called to Fig. 22, which shows an assumed set of pressure curves for a gravity dam. We can then proceed in the usual manner, i.e., divide the profile into a number of horizontal strips, calculate the resultants for each strip, and then draw the line of resistance for the entire dam.

To give an example, we shall examine the shaded strip 1234 (see Figs. 22 and 23). The equal-pressure lines divide this strip into a number of sections: lozenges, trapeziums and triangles. Let a_1, a_2, a_3 be the areas of these sections, as determined for instance by the planimeter, and let t_1, t_2, t_3 represent the average distances between the equal-pressure lines, measured to scale on the drawing. The formulae:

$$U_1=\frac{fa_1\triangle p}{t_1}\ ;\quad U_2=\frac{fa_2\triangle p}{t_2}\ ;\quad U_3=\frac{fa_3\triangle p}{t_3}$$

yield the individual interstitial pressures applied to the different sections of the strip. The symbol $\triangle p$ represents the difference in the interstitial pressures corresponding to two consecutive pressure curves, whereas f as before, is the coefficient of reduction. In other words, the part of the water pressure proportional to $1-f$ is still regarded as acting on the face of the dam.

As shown in Fig. 23, the resultant of all the forces thus applied to the strip under consideration can be found by means of a stress diagram and a funicular polygon or by any other method at the discretion of the computer.

In this manner the individual resultant will be found for every strip, and the computation of the line of pressure can then be carried out as simply as under the more usual, current assumptions.

It is not the author's intention, however, to recommend the stress-analysis method based on pressure lines for the daily work of the designing office; because in the practical routine case the normal stresses on horizontal planes (which are usually considered as the chief criterion of safety) will be the same whether they are calculated in the usual way or by the pressure-curves method. Shearing stresses, also, will remain equal, but the normal stresses on vertical planes will be changed.

This last point can be demonstrated by reference to the two profiles shown in Fig. 24. Let 13 and 46 be two horizontal planes drawn at an infinitely small distance *dh* from each other, and let us assume that in the left-hand section the hydraulic pressure is applied to the face of the dam, whereas in the right-hand section, this pressure is distributed over the entire masonry work. It will then follow that in the left-hand diagram the force acting on the surface *ab* is the external pressure minus the difference of the shearing stresses represented respectively by the areas 48*b* and 17*a*; whereas in the second case, instead of the force *pdh* we shall have that part only of this force which is distributed between 14 and *ab*, minus, again, the same difference of shearing stresses. It follows that the stress will be smaller in the second case, the difference being equal to that part of the hydrostatic load which is distributed between *ab* and 36.

This conclusion is interesting, for it may throw a new light on Professor Karl Pearson's theory, which in the earlier years of this century caused so much trouble to dam designers. In fact, several dam projects were then given up, and the first heightening of the Aswan dam was postponed, for the sole reason of Pearson's theoretical objections. His theory was that a dam fails, or should fail, because of tension developed on vertical planes, as in Fig. 25*A*, rather than on horizontal sections, as in Fig. 25*B*. It took much ingenuity, perseverance, and scientific ability to restore faith in the standard design methods, confidence in which had been so badly shaken by Pearson's iconoclastic attacks.

In fighting the battle, particularly important were the model tests carried out, under Prof. Unwin and Sir Benjamin Baker's general direction, by Wilson, Gore, Brightmore and others; but since the arrangement of the devices used in these experiments to simulate the effect of the water pressure conformed with the older views on the subject, it becomes now of some interest to verify whether the reduction in the compressive stress on vertical planes, consequent on the recent alterations in the assumed manner of action of the water pressure, can reverse the conclusions derived earlier from these tests.

In order to solve this problem, a special calculation was carried out in the author's office incorporating the new conceptions about the action of the water pressure. Some of the results are shown in Fig. 26. The profile investigated is the same as that used by the author in his earlier publications on shearing stresses. Three vertical sections near the tail of the dam are investigated, under different assumptions regarding the values of the coefficient f. With $f=0$ we have the older assumptions. The other limit, i.e., $f=1{\cdot}0$, represents the case when the entire water load is distributed over the masonry work. As will be seen from the drawing, the resultant remains for all cases well within the limits of the middle third. It follows that we should have no fears in respect to the safety of the vertical planes.

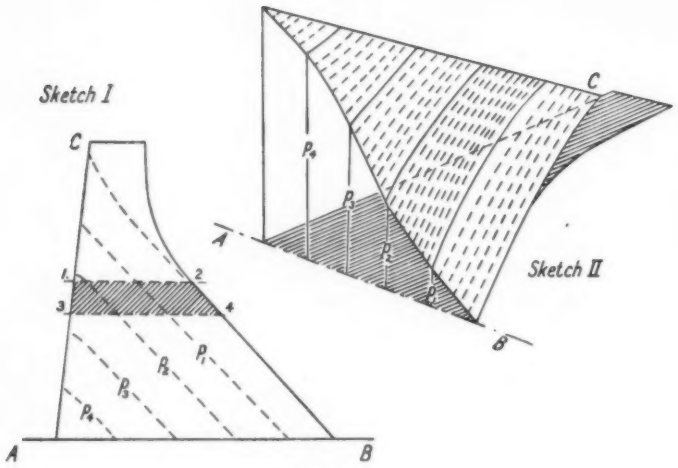


Fig. 22

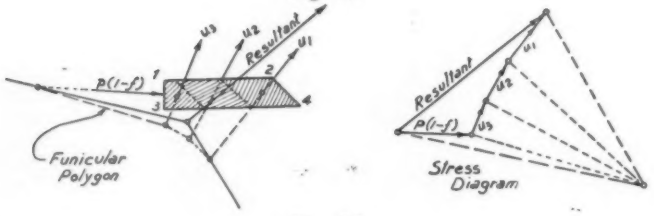


Fig. 23

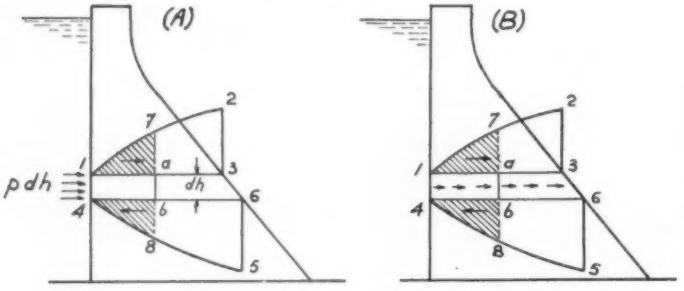


Fig. 24

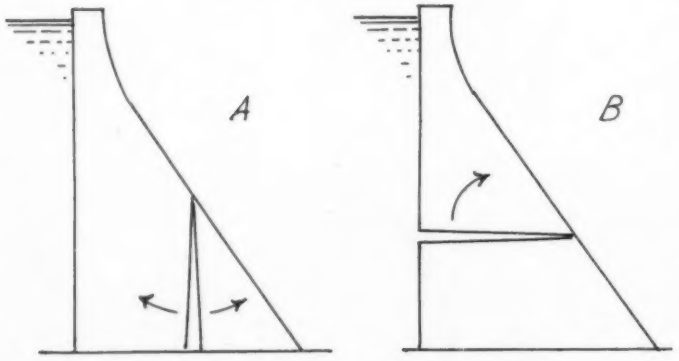


Fig. 25

It was said earlier that, in calculating the stresses under the assumption of distributed action of the water load, we may either use the lines of equal pressures, or the equipotentials. We have explained the methods of calculation based on the lines of equal pressures. Some words will now be added about Terzaghi's differential equations which, as will be seen from the following, are based on equipotentials. Fig. 27 represents a unit element of masonry or concrete, delimited by several physically possible surfaces of rupture, as conceived by Prof. Terzaghi and as shown earlier in Fig. 10*B*. Such surfaces, it will be remembered, while generally regular, incorporate, nevertheless, a maximum area of pores. In order that the flow of water in the pores should be steady, the reactive force R exerted by the walls of the pores upon the water which fills them must be equal and opposite to the active force A applied to this water. Hence, the equation

$$A = -R$$

We also know that the action is always equal to the reaction. It follows that the force A_1 exerted by the water upon the porous material, is equal and opposite to R; accordingly

$$A_1 = -(-R) = A$$

This is Terzaghi's basic solution. It means that in order to obtain the force transferred to the porous material by a certain volume of filtering water, it will suffice to find the resultant of all the active forces applied to this volume.

Consider, now, the unitary element *abcd* in Fig. 27. The surfaces by which it is delimited are supposed to be drawn following two flow lines and two equipotentials. The interstitial pressures on each side of the element depend upon, and are given by, the heights of the water columns in the respective piezometers (imaginary). For instance, for the surface *ab* the height of the column, which we may also describe as the hydraulic head, is equal to H, corresponding to a pressure of $H\rho$. To find the pressure on the surface *cd* we take into account the loss of head $\Delta H = s$ and the difference of levels at the bottoms of the manometers, $1{\cdot}0 \cos \alpha$. Hence the head for *cd* is $H - \Delta H + 1{\cdot}0 \cos \alpha$ and the corresponding pressure $(H - s + \cos \alpha)\,\rho$. Let f_s represent the ratio of the area of voids to the total area of Terzaghi's surface of failure. The difference of the total forces applied respectively to the faces *ab* and *cd* will then be

$$p_s - p'_s = H\rho f_s - (H - s + \cos \alpha)\rho f_s = (s - \cos \alpha)\rho f_s$$

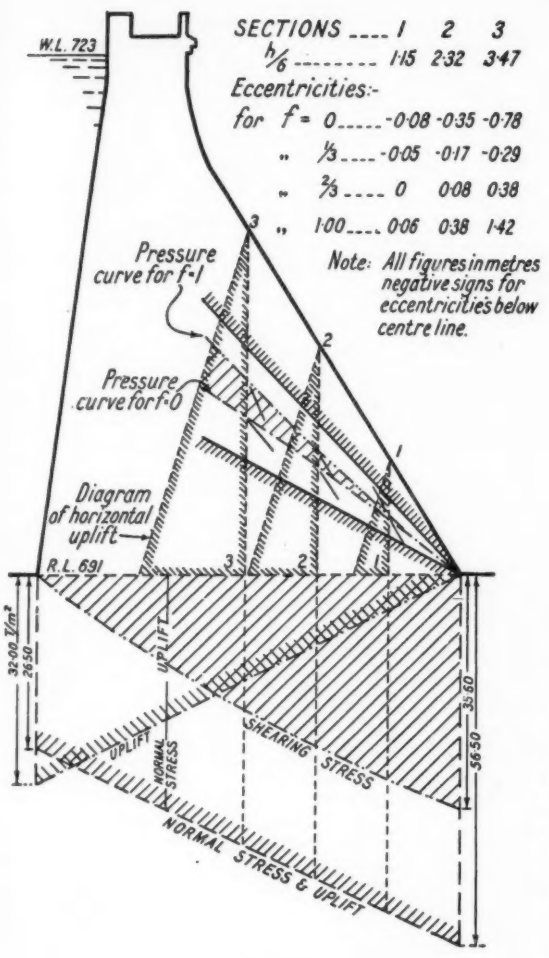


Fig. 26

This difference is a force which acts in the falling direction of the hydraulic gradient and is tangential to the line of flow.

In the same manner, in analysing the pressures applied to the faces of the element *ad* and *bc*, we shall find that the difference is equal to

$$p_n - p'_n = \rho f_s \sin \alpha$$

which is another force, perpendicular to the lines of flow and parallel to the equipotentials.

The third force is the weight of the water, viz. pλ.

The resultant of these three forces is capable of being resolved into two components:—

(a) a vertical component

$$F_a = f_s\rho - \lambda\rho = \rho\,(f_s - \lambda)$$

(b) an inclined force tangent to, and convergent with, the stream lines

$$F_s = \rho f_s s$$

According to Terzaghi's basic principle, these two forces represent the total load transferred by the filtering water to the elementary volume *abcd* in Fig. 27. It remains only to integrate them in order to obtain the total force acting on the structure.

If this method is applied to the triangular profile of Fig. 13, it will follow that the vertical force F_a represents a reduction in the weight, whereas F_s,

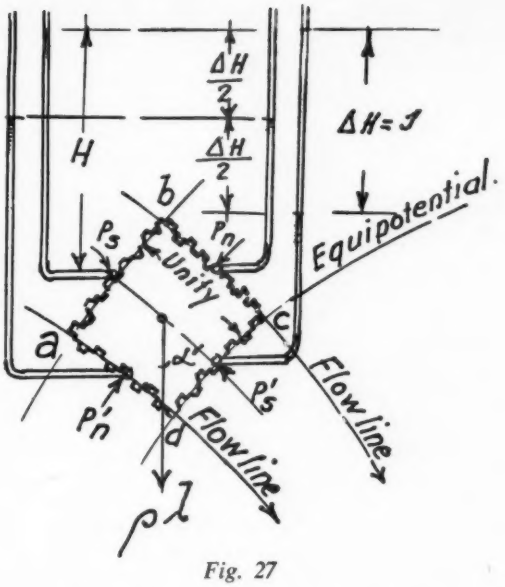


Fig. 27

being parallel to the flow lines is in this case horizontal. Hence, the two components, F_a and F_s represent here respectively the vertical and horizontal uplifts.

Another most important conclusion to be derived from the foregoing analysis is that in calculating the effect which the filtering water produces on the dam, we must always introduce the coefficient of reduction f_s, the remainder of the hydraulic load, proportional to $1-f_s$, being then applied to the face of the dam. This in fact is a point that has been frequently misunderstood; for instance, one of America's leading authorities on the subject, Mr. Roy W. Carlson, in a paper published as late as 1955, says:— "It can be proved that no matter what the pore area may be in the regions where pore pressure drops to zero, as long as that pressure does drop to zero, the uplift on a section at mid-length is the same." This statement, it will be realised disagrees basically with the argument of Fig. 27, and cannot be agreed with.

Uplift Pressure versus Uplift Stress

Attention will next be called to another fallacy which reappears from time to time in the publications of even the most advanced designers.

We refer here to a rather frequent misinterpretation of the nature of the concept itself: interstitial or uplift pressure. In fact designers are apt to forget that this is by no means an internal stress, in the modern sense of the term, but only a loading, i.e., an external force; and therefore, adding or subtracting them (i.e., interstitial pressure and internal stress) is a procedure as unorthodox as if we were, shall we say, multiplying an omelette by a steam engine, or dividing a steel nail by a memorial service.

The source of this rather popular error is not difficult to trace. It lies in the fact that in the case most often met with in the everyday practice, viz., an ordinary gravity dam, the cause (i.e., the pressure) is numerically equal to the result (i.e., the internal unit stress); which means to say that in so far as numerical results are concerned—but in so far only—adding and subtracting these two heterogeneous values is indeed admissible. The scope of this, possibly surprising,

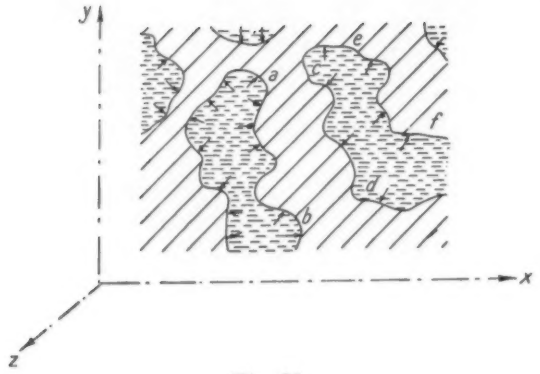


Fig. 28

conclusion, however, is confined to the case referred to. In fact let us suppose that the compressive stress at the heel of an ordinary gravity dam calculated under usual assumptions without taking uplift into account, is η, whereas the vertical uplift pressure at the same point is u. The office computer subtracts the one from the other and obtains directly, as final result, the stress $\eta - u$. Now, had he been following the true, academic path, he should have first found the total uplift force and the eccentricity, and then only, calculate the corresponding stress, as follows: for a triangular uplift distribution the uplift force would have been $bu/2$. and the eccentricity $e = b/6$. Hence, the stress would be found to be

$$\frac{N}{b}\left(1+6\,\frac{e}{b}\right)=\frac{bu/2}{b}\left(1+6\,\frac{b/6}{b}\right)=u$$

Paradoxical though this might appear, it follows that the computer was quite right in using a "wrong" method, but this is a purely coincidental result, the application of which should by no means be generalised, for uplift pressure is a concept basically distinct from uplift stress.

The importance of this conclusion can scarcely be overestimated in studying the uplift problem in general. For instance, it has sometimes been stated that uplift pressure cannot accumulate and produce an overall pull in the concrete. This statement, it will be observed, may lead to an entirely false conclusion. Although it is indeed an obvious fact that the pressure cannot accumulate, the stress due to this pressure can,

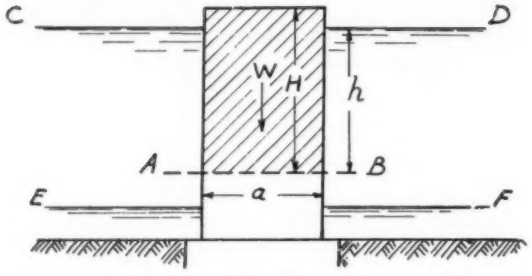


Fig. 29

and does, accumulate, thus creating an overall pull in the concrete. It is therefore essential to distinguish between uplift pressure and uplift stress, and to keep this difference always in mind, when dealing with relevant problems.

Nature and Value of the Reduction Coefficient

In the previous articles frequent reference has been made to the coefficient f without, however, producing the reasons justifying its use and explaining its significance. This will be the subject of the present section.

The first and possibly most important point in the relevant branch of mechanical theory refers to the relative importance of the effect of a constant pore-pressure as opposed to a variable one. Fig. 28 represents, to a greatly exaggerated scale, the microstructure of a porous solid impregnated with water. The arrows in the drawing represent the hydraulic pressure acting on the walls of the pores. This pressure may either be a constant for the entire block, or it may vary as a function of the co-ordinates x, y and z. In general, it can be conceived as the sum of a constant p_1=const. and a variable $p_2 = F(xyz)$.

The problem of the relative importance of p_1 and p_2 was attempted and solved by Prof. Fillunger in a strikingly simple manner, as early as 1915. His experiments were conducted with the common type of 8-shaped specimens usually employed for tensile tests on cements. Some of a set of identical specimens were tested to breaking point in free air, and others in a reservoir filled with water and subjected to a given pressure (so that the water penetrated, under pressure, into the pores). As seen from the following figures, the differences in the breaking loads were found to be negligible:—

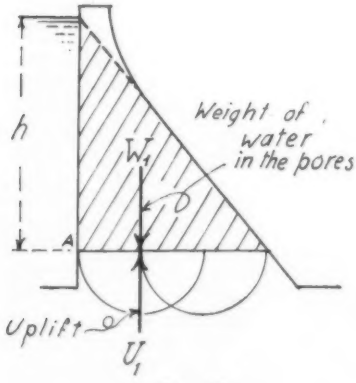


Fig. 30

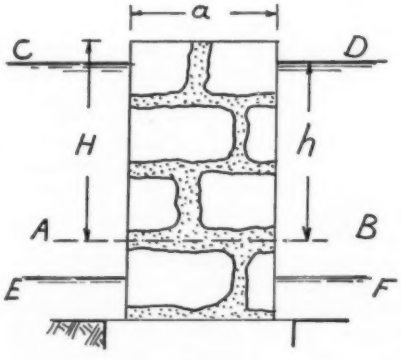


Fig. 31

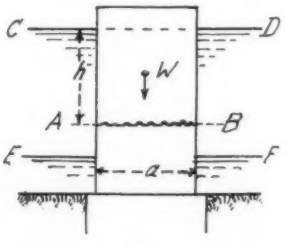


Fig. 32

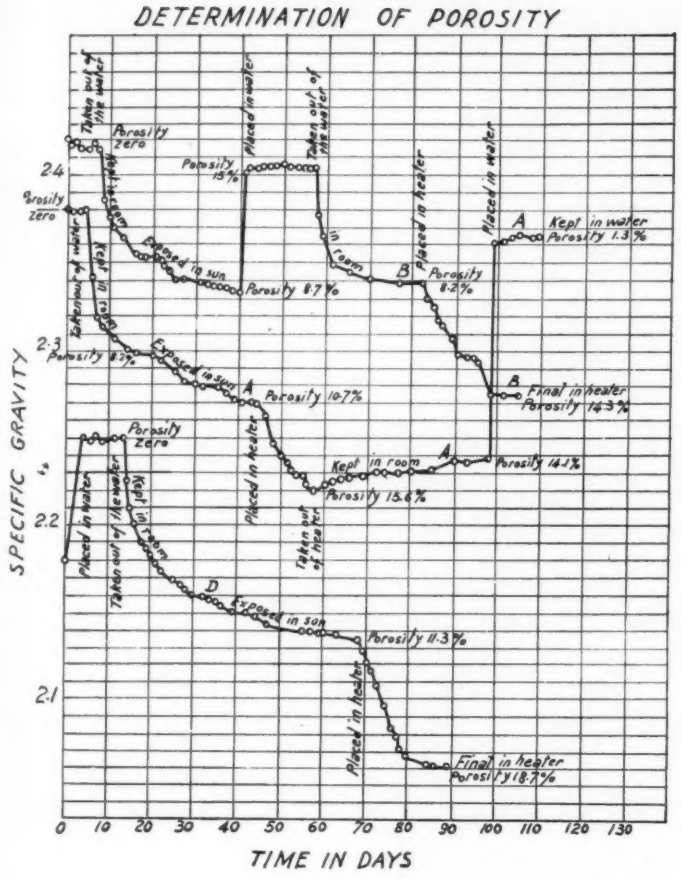


Fig. 33

Pressure in Reservoir	0	100	200 atm.
Breaking Load	36·1	32·4	31·6 kg. per sq cm.

This shows that the interstitial pressure in the pores, so long as it remains constant, has no effect on the stability or resistance of the material. In fact, so long as the pressures on either side of a solid, such as *ab* and *cd* in Fig. 28, remain equal, they balance each other, the only result being that the solid part *abcd* is compressed. This, as seen from the above figures, does not affect the porous material in any way whatever. Such a pressure is therefore described by Prof. Terzaghi as "neutral pressure." In more recent times, i.e., in his experiments of 1934, Prof. Terzaghi has shown that the conclusion derived by Fillunger from tensile tests holds also good for compression tests. It follows that whether concrete is tested in free air, above ground level, or down below, at the bottom of the sea, the results will always be the same.

Another important point belonging to the same group of problems demonstrated by Fillunger was that the coefficient f is really the difference of two factors: a positive surface factor f_s and a negative volume factor f_v. As Fillunger's own method for demonstrating this theorem is rather involved, we shall use instead Terzaghi's much simpler method, as follows. Consider the pier shown in Fig. 29; so long as the water level stands below the section *AB* (for instance at *EF*) the stress in that section depends solely on the weight of the material above it. Thus

$$\frac{W}{a} = Hv$$

in which W is the weight of the pier above AB,
a is the width of the pier,
H is the height of the pier above AB
and v is the specific weight of material.

Should, however, the water level rise up to *CD* the stresses must change, the effect being essentially twofold:—

(a) the pore pressure acting upwards will relieve the compressive stress. If we assume that this relief is equal to the pressure multiplied by the area of pores, its value will be $h\rho a\lambda$. The unit stress will then be reduced to

$$Hv - \frac{h\rho a\lambda}{a} = Hv - h\rho\lambda$$

(b) Owing to the water which has penetrated in the pores, the weight of masonry will augment. This additional weight will be $ha\lambda\rho$, and the unit stress will therefore be increased by $h\lambda\rho$.

It follows that the total stress after the rise of the water level will be

$$Hv - h\lambda\rho + h\lambda\rho = Hv$$

which means that it will not change at all.

The method of this demonstration can be shown to apply as well to a gravity dam. Take, for instance, the profile shown in Fig. 30. Suppose that the stress at the point *A* calculated under the usual assumptions of an impervious dam is η_1. For a porous dam we shall have to introduce the following alterations:

(a) An uplift force U_1, which is applied at the middle third, and is presumably equal to

$$U_1 = \frac{h^2\lambda\rho \tan \alpha}{2}$$

(b) The additional weight of water in the pores W_1, which is applied in the same point and is also equal to

$$W_1 = \frac{h^2\lambda\rho \tan \alpha}{2}$$

but is directed downwards.

Hence, the result of these alterations is zero. This conclusion may possibly appear surprising. The "uplift force," for so many years the most debated topic in dam engineering, appears, in fact, to be non-existent, and the stress still remains the same. It will, however, be easily understood that this paradoxical conclusion does not apply to the uplift force itself—which is very real indeed—but to the assumption $f=\lambda$ taken as granted in these calculations. In fact, we calculated the uplift force by multiplying the uplift pressure by the porosity coefficient λ, and this was the true reason that led us to absurd results.

Since this approach is now shown to be obviously wrong, the problem must be solved on another basis.

Three solutions deserve particular mention:—

(1) Fillunger's method, based on the difference between the average porosity of the masonry and the porosity of mortar.

(2) Terzaghi's solution, derived from his concept of a specific "surface of failure."

(3) The author's purely empirical approach.

To commence with Fillunger's method, instead of Fig. 29, let us consider the pier shown in Fig. 31, and investigate the plane section AB passing through a joint. The upward uplift force will then be $h\rho a\lambda_1$, where λ_1 is the porosity of hardened mortar; whereas the weight of additional water in the pores will still be $h\rho a\lambda$, and since by postulate $\lambda_1 + \lambda$, the total uplift force will be $h\rho a(\lambda_1 - \lambda)$. Thus, the uplift coefficient is found to be $f = \lambda_1 - \lambda$.

This solution is a material improvement on the fallacious principle embodied in the elementary formula $f = \lambda$. But all things considered, it is hard to believe that the uplift force, which is a significant factor in the design, depends solely and exclusively on a secondary consideration of rather uncertain value and questionable significance.

Terzaghi's solution is schematically represented in Fig. 32, which shows the same pier as in Fig. 29, but instead of a plane section contemplates the case of a failure along the surface of maximum porosity (as represented earlier in Figs. 10*B* and 27). Since this surface is by assumption generally regular, the additional weight of water in the pores will still remain $h\rho a\lambda$, but the uplift force will now be many times larger, viz., $h\rho af_s$, in which f_s is the "effective porosity" for the specific type of "surface of failure" here considered. It can only be determined by test, but is obviously much greater than λ. Hence, the net uplift force becomes equal to $h\rho a(f_s - \lambda)$, and the coefficient $f = f_s - \lambda$ is therefore equal to the difference between a positive surface factor, f_s, and a negative volume-factor λ.

The author's solution is a purely empirical approach. It may be transcribed in the form $f = f_s - f_v$ in which both f_s and f_v must be found experimentally.

Taking first the volume factor f_v, it will be observed that at the moment when tests are carried out in the field in order to find the specific weight of the material which is to be used for calculating the dam, the pores are already partly filled with water. The addition in weight, when later on the material is fully impregnated, is therefore much smaller than that calculated according to the ideal porosity λ. It must consequently be determined by test.

In Figs. 33 and 34 we have the author's porosity tests carried out with various materials, under different conditions. The specimens were abstracted from the author's uplift experiments, where they had been impregnated with water under pressures up to 300 tons per sq. m. Determinations of porosity were continued daily for several months. This period was subdivided into four operations, as follows: (a) specimen submerged in water; (b) specimen kept in the room; (c) specimen exposed to the full action of wind and sun; (d) specimen placed in heater. In some cases the operations were repeated, but the results were always the same: the maximum porosity never exceeded 15·6%, with an average of about 7·0%. This last figure is recommended by the author for practical application. On the other hand, the author's empirical method for determining the surface factor f_s will be given in detail in the forthcoming article.

(To be continued)

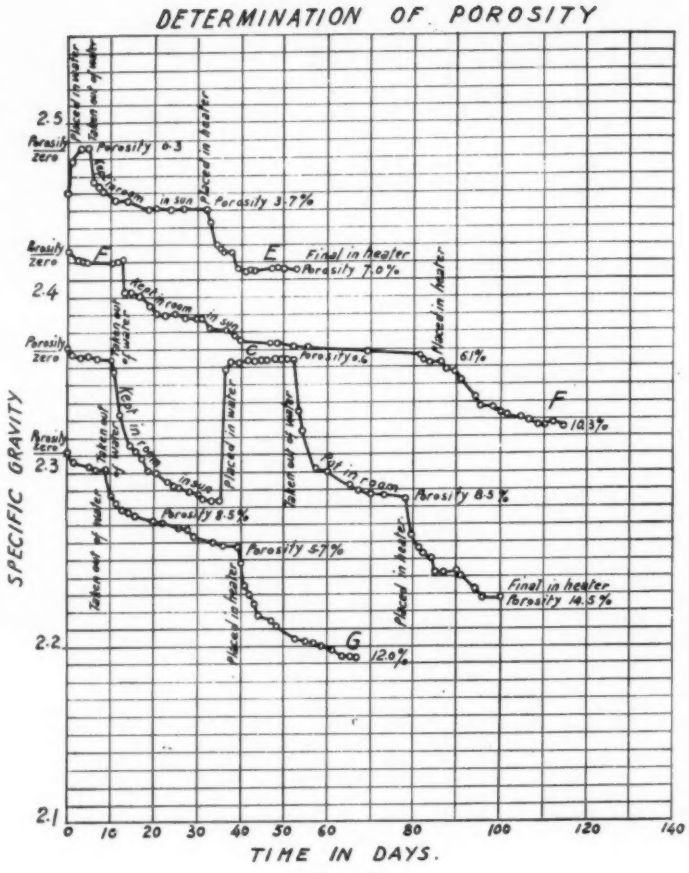


Fig. 34

Cold Caulking Compounds. A pamphlet received from Philplug Products Limited, Wembley, Middlesex, refers to an asbestos/cement caulking material for jointing concrete tunnel segments that will withstand pressures of 1,000 lb. per sq. in. or more.

Concrete Mixers and Water Pumps. Ransomes & Rapier Limited have sent us a copy of their Publication No. 1/528 which refers to their truck-type concrete mixers and agitators, and their self-priming water pumps which have outputs ranging from 4,800 to 96,000 U.S. g.p.h.

Norway's Hydro-Electric Power and Industries

An economic study of Norway's power potentialities and development, and of the industries which the country's cheap power has brought into being

PART ONE

THE purpose of this article is to describe the current and future development of Norway's cheap hydro-electric power resources and the many electro-chemical and metallurgical industries which thrive thereon. Installed generating capacity has doubled since the war and now exceeds 5,000 MW. Undeveloped resources would permit of at least another 10,000 MW to be economically installed. The present price of industrial power averages less than a farthing a unit. It is cheaper on the average than any power in Europe or even in North America; it is one-quarter to one-fifth the price of industrial power in the United Kingdom. It is likely to remain cheaper than any other form of power for many years to come; perhaps by 1990 the rising cost of additional power in Norway will meet the falling cost of nuclear electric power. Substantial quantities of raw and finished fertilisers of all kinds, hydrogen, ammonia, "heavy water," chlorine, chlorates, calcium carbide, aluminium, ferro-alloys, pig iron and steel, zinc, nickel, magnesium, and smaller quantities of many other valuable metals or chemicals are being produced by electrolysis or electric smelting in Norway and being exported. Their production and that of the power on which they are based, is rising rapidly. Norway, however, is short of capital and would welcome foreign capital which would assist and speed up this expansion.

Growth of Hydro-Electric Power Capacity

With her rugged mountains, deep valleys and heavy rainfall, exceeding 80 in. a year on the west coast, Norway is especially well endowed with hydro-electric resources. Furthermore, she is impelled to make the most of them, possessing as she does only limited quantities of soft coal in Spitzbergen and no oil. A few small hydro-electric plants were already installed by the end of the nineteenth century, but these, as in other countries at the time, were largely experimental and were harnessed to slow-flowing rivers in the eastern and central parts of Norway. In due course, with technical advances in rock-blasting and water-turbine design, high-head falls began to be harnessed. Today nearly all new hydro-electric plants are linked to high-head falls. (An exception is the planned exploitation, jointly with the Russians, of the low-head Pasvik River on the Russo-Norwegian border.) By 1910, the installed generating capacity of the country was about 250 MW. As will be seen in the graph in Fig. 1, by 1938 this had risen to over 2,000 MW and by 1945 had reached 2,500 MW. Now, 14 years later, this latter figure has been more than doubled. Work has already begun, or has been fully planned and is about to start, on plants that will add another 2,000 MW over the next four years. This will bring the total installed capacity up to about 7,500 MW or about 2 kW per head of population by 1963.

As will likewise be seen from Fig. 1, the amount of power actually generated has increased roughly in proportion to the capacity installed. In 1958 some 27,000 million kWh were generated or about 7,500 kWh per head of the population of three and a half million. It compares with a total thermal and hydro-electric production in the United Kingdom of about 1,500 kWh per head in the same year. The increase in production since 1938 of about 8% per annum is comparable with the concurrent rate of increase per head in North America, Sweden or Switzerland.

Further Undeveloped Potentialities

How much more generating capacity beyond 7,500 MW can be economically installed in Norway? The answer to this question depends on a great many factors and experts are continually revising their estimates. The last official estimate of Norway's total resources was prepared for the World Power Conference in 1955. Based on hydrographic data, Norway's total potential resources were then put at 12,500 MW which would call for an installed generating capacity of nearly 22,000 MW. This suggests that even after

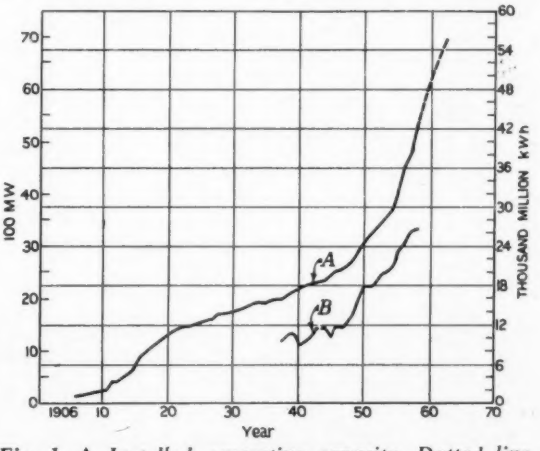


Fig. 1. A *Installed generating capacity. Dotted line indicates new capacity planned or under construction.* B *Total electricity production*

the present generating programme has been completed, a total of almost 15,000 MW generating capacity could in principle still be installed in the country. This estimate, moreover, does not take into account the several large rivers and water catchments on the Swedish side of the frontier, which it might be economic to divert and develop on the Norwegian side. Actually the Norwegian-Swedish-Finnish joint committee has suggested building a 1,000 MW power station on the Rombaks Fjord, near Narvik, drawing water from Sweden. Table I shows by county or region the capacity installed in Norway up to December, 1958, the total capacity due to have been installed by the end of 1963 in accordance with present plans (these may well be augmented later), the total potential capacity as estimated in 1955 and the balance of undeveloped capacity beyond 1963.

As the Table shows, both the developed and undeveloped capacity is widely, if unevenly, scattered over the whole country, but the largest amount of undeveloped capacity lies in West Norway, the area behind the main ports of Stavanger and Bergen, from which there are good sea and air services to the United Kingdom and the Continent. Relative to the size of the local population, potential capacity is being developed fastest in North Norway, where the population is only half a million. This is because the need for power is greatest in those parts for geographic, climatic and, above all, for social reasons. For these same reasons the Government gives industries establishing themselves in North Norway special tax concessions and other forms of assistance.

Cost of Developing Resources and Sale Prices

The cost of developing Norway's hydro-electricity resources has naturally varied over time and with the location of the resources developed, and will continue to do so. The cheapest or easiest resources have naturally tended to be developed first, and as time passes and more of the resources are developed, the remaining resources, as a general rule, are more expensive to develop. The development of hydro-electricity resources, in other words, tends to be subject to diminishing returns. The cost of development has also inevitably risen in proportion to the depreciation of the currency and the rise in labour costs. Before the last world war water reservoirs and power plants were being constructed in Norway at a cost of about £20 per kW, and in the twenties and early thirties the cost was lower than that. Costs of developing the three large recently completed projects, Aura (280 MW), Rössåga (240 MW) and Tokke (400 MW)—excluding transmission lines, have been put respectively at £13·5 million (£48·2 per kW), £11·35 million (£47·3 per kW) and £19 million (£47·5 per kW).

The cost of developing Norway's hydro-electric resources will inevitably continue to rise, but almost certainly they will also continue to be more economic than thermal stations, fuelled with either coal or oil, at least in Europe. Competition from nuclear power plants is admittedly looming on the horizon, but according to estimates made in 1957 by Sir Christopher Hinton and Professor Vogt, Director-General of the Norwegian Hydro-Electricity Authority, rising costs of power from new *marginal* hydro-electric plants in Norway are not likely to meet the falling costs of power from new *marginal* nuclear plants before 1990. A graph drawn up by Professor Vogt in 1957 projecting his estimates of the rising cost of Norway's hydro-electric power against Sir Christopher Hinton's estimates of the falling cost of nuclear power is reproduced in Fig. 2.

The price of power to consumers, which is inevitably higher than the cost of generation, has also risen gradually in Norway, but not in direct proportion to the rise in capital costs. As in other countries, Nor-

TABLE I.—INSTALLED, PLANNED AND POTENTIAL HYDRO-ELECTRIC CAPACITY IN NORWAY (MW)

County	Installed capacity at end of 1958	Due to be installed by end of 1963	Total capacity by end of 1963	Total potential capacity	Balance of undeveloped capacity
East Norway					
Østfold (inc. Oslo) ...	440	27			
Hedmark	29	3			
Oppland	358	182			
Buskerud	801	183	3,445	6,100	2,655
Telemark	926	491			
Vestfold	5	—			
South Norway					
Aust-Agder	62	20			
Vest-Agder	273	17	372	1,900	1,528
West Norway					
Rogaland	300	80			
Hordaland	548	120	1,620	7,900	6,280
Sogn and Fjordane ...	322	250			
Central Norway					
More and Romsdal ...	460	56			
Sør Trondelag ...	126	188	1,169	2,500	1,331
Nord Trondelag ...	121	218			
North Norway					
Nordland	556	160			
Troms	43	113	976	3,500	2,524
Finmark	28	76			
Total	5,398	2,184	7,582	21,900	14,318

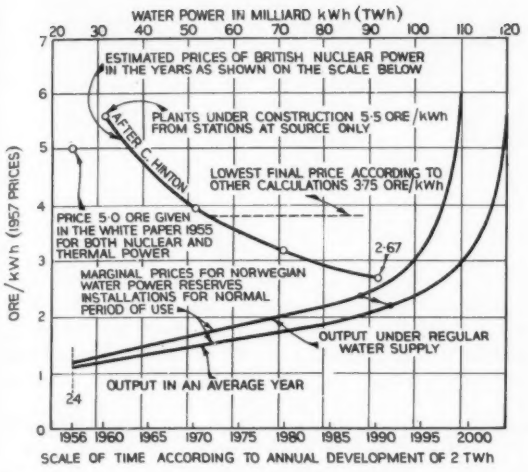

WATER POWER IN MILLIARD kWh (TWh)
20 30 40 50 60 70 80 90 100 110 120
ESTIMATED PRICES OF BRITISH NUCLEAR POWER IN THE YEARS AS SHOWN ON THE SCALE BELOW
PLANTS UNDER CONSTRUCTION 5·5 ORE/kWh FROM STATIONS AT SOURCE ONLY
AFTER C. HINTON
LOWEST FINAL PRICE ACCORDING TO OTHER CALCULATIONS 3·75 ORE/kWh
PRICE 5·0 ORE GIVEN IN THE WHITE PAPER 1955 FOR BOTH NUCLEAR AND THERMAL POWER
2·67
MARGINAL PRICES FOR NORWEGIAN WATER POWER RESERVES INSTALLATIONS FOR NORMAL PERIOD OF USE
OUTPUT UNDER REGULAR WATER SUPPLY
OUTPUT IN AN AVERAGE YEAR
24
ORE/kWh (1957 PRICES)
0 1 2 3 4 5 6 7
1956 1960 1965 1970 1975 1980 1985 1990 1995 2000
SCALE OF TIME ACCORDING TO ANNUAL DEVELOPMENT OF 2 TWh

Fig. 2. Time scale based on increase in hydro-electricity production of 2,000 kWh p.a. Upper curve shows estimated fall in British nuclear power prices; lower curves show the price increase of Norwegian hydro-power as poorer resources are utilised. Price equation cannot be estimated until all water sources have been tapped

wegian power tariffs are not homogeneous, and domestic consumers pay more than industrial consumers. If individual industrial units have not developed power concessions of their own, they usually conclude long-term power contracts with the central or local authorities at fixed rates or only partly at sliding scales. Thus at any one time a whole range of different tariff rates will be operating throughout the country. Some industrial units in Norway still enjoy tariffs, contracted shortly after the last war, of less than 0·125d. per kWh. More recently, new contracts have incorporated tariffs of between 0·18d. and a farthing. There is a fair prospect of new industries in Norway, over the next few years, still being able to conclude long-term contracts for as little as about a farthing a unit.

Power Cost Comparison with Other Countries

The cost of hydro-electric installations and the price of power to industrial consumers are clearly very much lower in Norway than in the United Kingdom, but how do they compare with those of other countries famous for their water-power resources? Exact comparisons of power costs or prices between countries are almost impossible because of the multiplicity of tariffs and their seasonal variations, but according to calculations of Norwegian experts, made in 1954 and based on data submitted by member countries to the Organisation for European Economic Co-operation in Paris, Norway's present costs or prices compare very favourably with those of Switzerland and Austria, and even with those of Sweden. According to an article by Professor Vogt, written in 1954, an average annual price of industrial power in Austria was about £18 per kW. For Norway he quoted a corresponding price of £4·5 to £5 per kW. For Switzerland he gave the generating costs shown in Table II. The average price to industrial consumers in Norway would be about 0·12d. for 1947 and about 0·18d.

TABLE II.—GENERATING COSTS IN SWITZERLAND
(Pence per kWh)

	Plant in service on 1.1.47	Commissioned later or in construction	In planning stage
Winter half-year:			
Weekdays, day time	0·55	0·86	0·8
Holidays and nights	0·36	0·52	0·52
Summer half-year:			
Weekdays, day time	0·31	0·49	0·48
Holidays and nights	0·14	0·22	0·22

to 0·24d. today. This serves to illustrate the comparative cheapness of electricity in Norway.

Sweden's hydro-electric resources although at present costing little more to develop than those in Norway are located in the north while her consuming demand is in the south. The price of her power has therefore to include high transmission costs. Her total potential resources, moreover, are less than Norway's and nearly all her "cheap" resources have already been built out. They are already supplemented by thermal power. According to a Swedish periodical quoted by Professor Vogt, the Swedish power development programme up to 1960 provides for the development of a number of new stations or the extension of old ones totalling 1,500 MW capacity. The cost of building four of the new stations should average not more than £40 per kW. The extensions of three existing power stations should average not more than £35 per kW. The programme also included, however, seven stations at an average cost of £65 per kW. The whole programme averaged about £50 per kW (excluding the very cheap Harsprånget plant). In addition at that time the Swedish authorities were contemplating the establishment of a new thermal 200 MW power station near Gothenberg, of which the capital and production costs were estimated to amount to about 1·61d. per kWh. That Sweden's development costs must from now on be considerably higher than Norway's is demonstrated by the fact that Sweden already buys power from Norway and is anxious to buy more.

Altogether it would seem safe to conclude from the figures quoted that a manufacturer contemplating establishing a new electro-chemical or metallurgical plant would find cheaper power in Norway than in any other country in Europe or even in North America. In the latter continent certain manufacturers have considered that there are other advantages which offset Norway's advantage of cheap power. This is a question which will be reverted to later. From the point of view of manufacturing and selling electro-chemical and metallurgical products, however, Norway obviously enjoys very great advantages compared with all the larger industrialised states of Europe.

(To be continued)

Boving & Co. Ltd. have moved from 56 Kingsway, London, W.C.2, to new and larger premises at Villiers House, 41/47 Strand, London, W.C.2 (Telephone: TRAfalgar 2401-6).

Steel Castings. Publication No. 5938 from Crofts (Engineers) Limited, Bradford, refers to the work this well-known firm does in regard to the manufacture of steel castings up to eight tons in weight.

Aschach Station

This power plant, which will be the largest on the Austrian Danube, will be the next in the chain to be constructed

By Dr. E. KÖNIGSHOFER

AFTER completion of the power stations at Jochenstein (WATER POWER, March 1957) and Ybbs-Persenbeug (WATER POWER, June 1956), the development of a third stage on the Austrian Danube has been initiated after the solution of very severe financial problems. The plant will be constructed near the village of Aschach, after which it will be named, and will be erected in accordance with the overall plan elaborated a few years ago for the development of the Austrian Danube by the Österreichische Donaukraftwerke A.G.

This statement, however, should not be interpreted to mean that the plan to utilise the Austrian Danube for energy production has only recently been conceived, because this project had been under consideration ever since the technique of run-of-river plant construction made such work feasible. As is known, the first big European run-of-river plants were built in Switzerland. The construction of the Rheinfelden, Laufenburg and Augst-Wyhlen power plants was begun more than 50 years ago, and the favourable experience gained from these stations suggested the development of the Austrian Danube. Actually, the first incentive to realise the Ybbs-Persenbeug stage of the Danube came from a Swiss source. The first studies made to ascertain whether the Austrian Danube was suitable for power production showed that the conditions there were very favourable. Over a distance of 350 km. the Danube has a gross head of about 160 m. and has a dependable flow having a very large catchment area, comprising 76,600 sq. km. when entering Austrian territory and 104,700 sq. km. when leaving it. The flow is very well balanced, the ratio between winter and summer flow being 43 : 57.

Recognition of the fact that the Danube is a very important potential energy producer entailed the danger that those interested in its development might choose the most profitable sections for power production and neglect the concerted overall development by which alone the maximum power production could be assured. That is why in 1946 the so-called "Danube Committee" was set up which, headed by Professor Dr. Oskar Vas, was entrusted with the preparation of an overall plan of development for the river; this plan has been worked out in detail by the Donaukraftwerke A.G., founded in 1947.

Fig. 2 and Table I give a survey of the stages envisaged, as well as the capacity and production of each section.

Among the above developments, the Jochenstein and Ybbs-Persenbeug plants are already known to the readers of WATER POWER. It should further be noted that the lowest stage of Wolfsthal is under consideration for joint development with Czechoslovakia.

The overall development plan elaborated in detail by the Austrian Donaukraftwerke A.G. provides for an 80% utilisation of the gross head of the Austrian Danube—a ratio that can be regarded as very high, higher, indeed, than that of any other known chain of run-of-river plants. This overall plan takes care of navigation interests, flood protection, fishing, water supply, etc. It is of special interest to navigation since various natural obstacles will be submerged and river transport substantially facilitated. After the complete execution of the overall plan Danube navigation will be effected on apparently stagnant storage water; the existing natural obstacles will have been submerged in water and river traffic will move upstream and downstream at almost the same speed, the movement of the water being only perceptible at weirs.

TABLE I.—ENERGY DATA OF THE VARIOUS STORAGE STAGES

Plant	Distance of weir from mouth of river	Top water level above Adriatic Sea level	Net head at medium flow	Water flow	Capacity	Annual production
	km.	m.	m.	cu. m./sec.	MW	Gwh
Jochenstein	2203·33	290·34	10·15	1750	140	870
Aschach	2162·67	280·00	15·66	2030	264	1609
Ottensheim	2145·50	264·00	8·44	1750	120	750
Linz	2131·00	254·50	5·68	1750	80	507
Mauthausen	2113·85	247·00	6·17	1800	90	574
Wallsee	2094·00	239·50	9·19	2000	147	992
Ybbs-Persenbeug	2060·42	226·20	11·03	2100	189	1252
Melk	2034·75	213·50	8·12	2150	137	907
Rossatz	2010·20	203·00	7·47	2150	126	838
Grafenwörth	1984·90	194·00	10·72	2150	182	1203
Tulln	1965·90	182·00	8·14	2250	143	952
Klosterneuburg	1939·00	171·77	10·06	2300	181	1206
Wien	1920·80	159·79	6·35	2300	102	636
Petronell	1890·00	152·00	10·40	2300	193	1280
Wolfsthal	1873·00	141·00	10·00	2400	192	1330

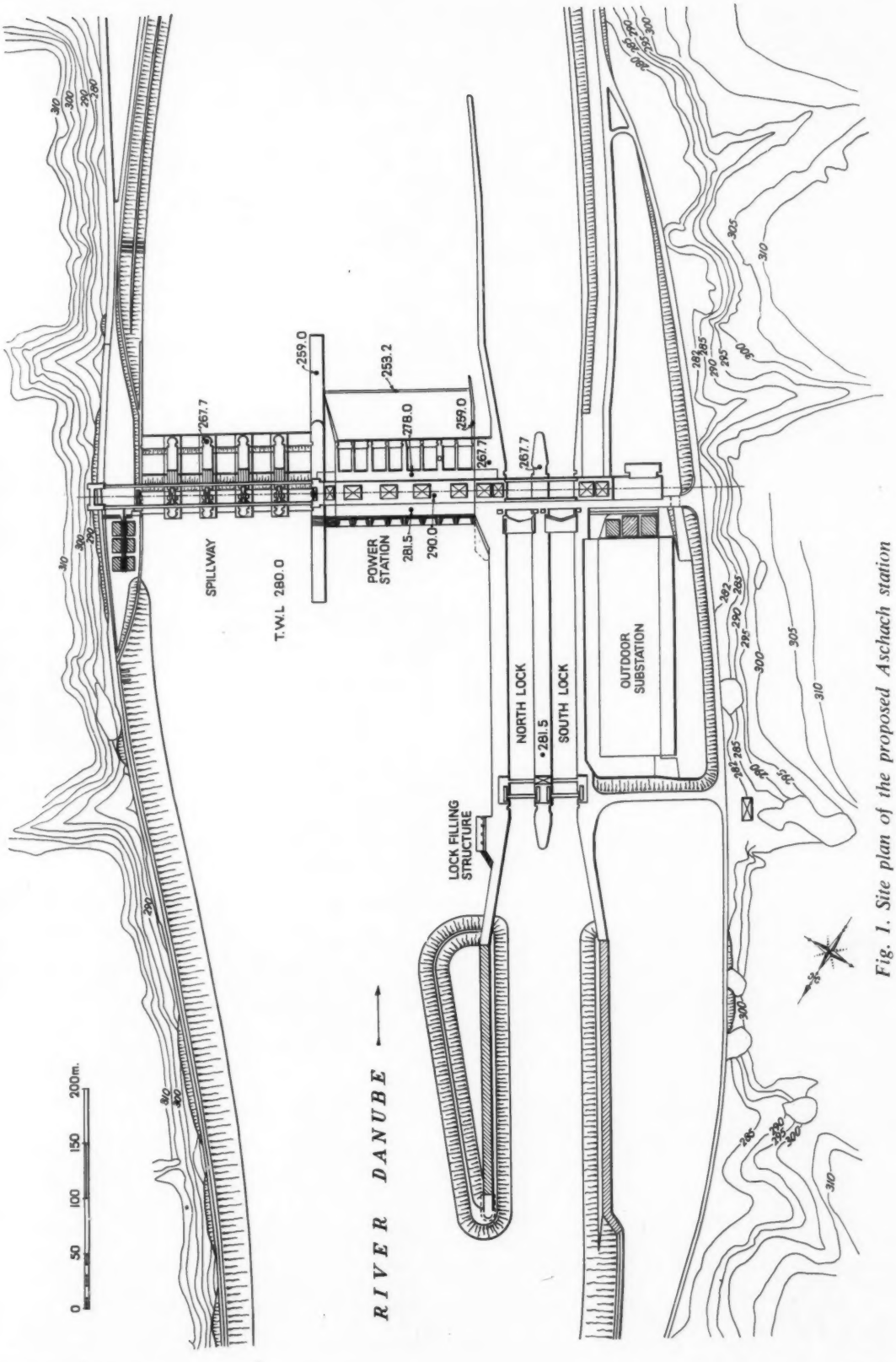


Fig. 1. Site plan of the proposed Aschach station

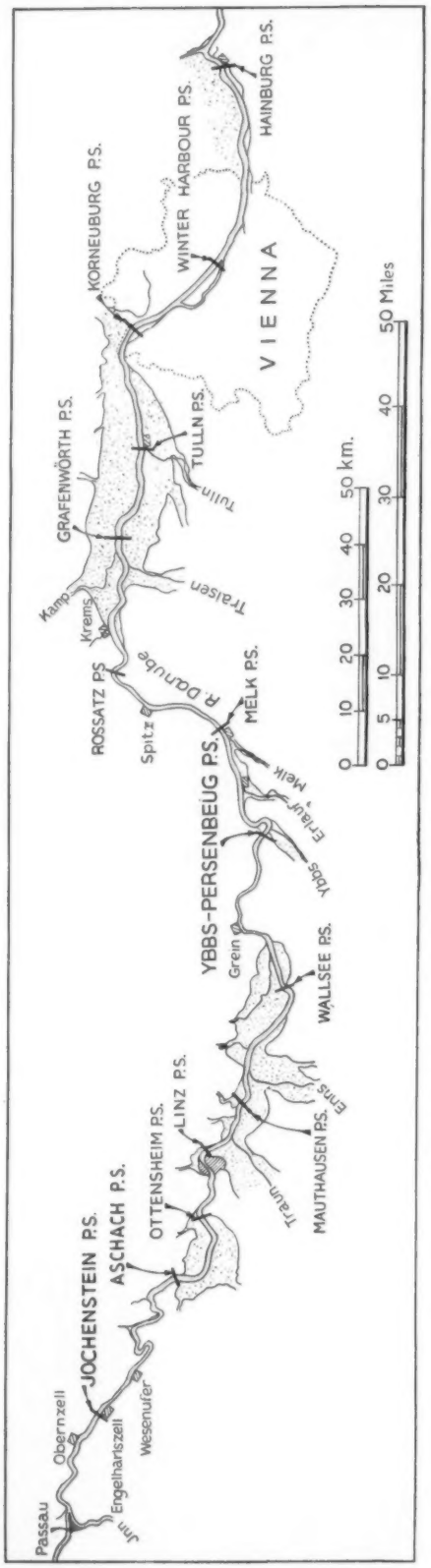


Fig. 2. Map of the Austrian Danube showing the proposed chain of power stations

The village of Aschach is one of those many picturesque places for which the Danube valley is so well known. The installations planned will not endanger the village, as the storage point will be situated upstream of the village, at the end of a narrow curved gorge. Local conditions at the power site are very favourable. The valley is sufficiently wide and the river course extends far enough to accommodate all parts of the plant including the weir installation. The geological and material supply conditions are also especially favourable there.

With the completion of the Aschach plant the head of the Danube down to the Jochenstein plant will be utilised. The storage reservoir will extend over a distance of 40 km., including very few low stretches and few settlements, as the river flows predominantly between steep mountain slopes. Detailed investigations and calculations have shown that it would be most expedient to raise the average water level by 15·66 m., i.e. to fix the maximum water level at 280 m., and in this way the water velocity will be considerably reduced. The narrow winding valley above the storage point will be turned into a long, deep mountain lake in which navigation can be effected without the risks hitherto entailed. Only one village on the left bank (Untermühl) will have to be completely resited, but some of the houses and the loading facilities in another (Obermühl) will have to be re-erected. In a few instances it will be necessary to rebuild farms situated close to the river bank, and to raise the level of low-lying areas. In one locality the embankment road will have to be rerouted at a higher level, and in the village of Nieder Ranna the river banks will have to be reconstructed. Two generating stations, both owned by the Oberösterreichische Kraftwerke A.G. (OKA), will be affected by the backwater. In the Partenstein plant the canal, tunnel and turbine draft tubes will be submerged to a depth of several metres, and the turbines and the tailrace will have to be raised to a higher level. Compensation will have to be provided for the 8·6% reduction of capacity resulting therefrom. In the Kramesau plant on the Danube an additional Kaplan turbine is operated at present only at low water discharge, which will no longer be available when the Danube is backed up. The OKA will have to be compensated for this loss of power production.

The backwater basin envisaged will affect the tailwater level of the Jochenstein plant and will probably result in lowering the present production in a normal year by 62 million kWh. Thorough investigation has shown that if the maximum water level at Aschach is decreased below 280 m., the loss in power production there would be much greater than at Jochenstein.

Turning, now, to a more detailed description of the scheme, the easier conditions for construction work obtaining on the right bank of the river at the site of the development led to the decision to locate the lock, power station and outdoor substation on the southern side. The lock itself is adjacent to the right bank, and adjoining it is the power house containing four main generating units and one auxiliary set. Then follows the spillway, which has five openings and is separated from the power house by a pillar 12 m. wide. The dam crest is at a level of 281·5 m., providing 1·5 m. of freeboard. Two crane bridges serve as a runway for two portal cranes, each having a carrying capacity of 180 tons. There is no public roadway over the dam.

Locks

From the installations erected on the Danube at Kachlet (Bavaria), Jochenstein and at Ybbs-Persenbeug a standard design has been developed which has also been adopted for Aschach. A twin lock with a usable width of 24 m. and a usable length of 230 m. has been provided. With its bays the lock installation will be 1,100 m. long. The headbay is 100 m. wide and 250 m. long, the tailbay 90 m. wide and 230 m. long. The upper gates consist of vertical-lift roller sluices, 15·5 m. high, the drives being mounted on tower-like superstructures on the lock walls. These superstructures are connected by covered footbridges, and both gates are operated from a control tower on the centre wall. The lower gates of both locks are in the form of the conventional mitregates. Experiments in filling and emptying the locks have shown that the high lift required and the large water volume involved make the system practised in other Danube plants of filling the lock from the headbay impracticable, and the filling water is therefore taken from the main river. On the river side of the northern wall there is a structure which connects with four culverts below the floor of each lock. These are equipped with filling slots spread over the whole chamber floor, and ensure that the lock chamber is evenly filled along its whole length. Emptying is effected in the same manner, the four culverts discharging to the tailbay in parallel with the turbine flow. Thus the ships in the lock are unaffected by the filling or emptying of the chamber. The lock chambers can be closed upstream and downstream by means of stoplogs and dried out.

Spillway

The spillway adjoins the power house on the far side from the lock. It is 156 m. long and consists of five openings, each with a free width of 24 m., and four piers each 9 m. thick. Detailed calculations and model tests have shown that it is admissible to place the spillway sill at 265 m. This relatively high level of the sill has the advantage that the dimensions of the movable gates are favourable and that the costs involved are not excessive. On the other hand, there is the disadvantage that the sill cannot be completed during the construction period as an inadmissible water level would be reached during the construction work and uncontrollable water velocities would arise.

The gates are of the hooked twin type and have a total height of 15·5 m., 0·5 m. of which is freeboard. By lowering the upper leaves fully without lifting the lower leaves, and by placing the turbines on full gate it will be possible to discharge floods of a size corresponding to any that occurred within the last 20 years. The gate drives are mounted on the weir piers.

The spillway openings can be closed on both sides by stoplogs, and to relieve the hydraulic pressure when drying out a spillway floor, drainage pipes will be inserted below the spillway foundations.

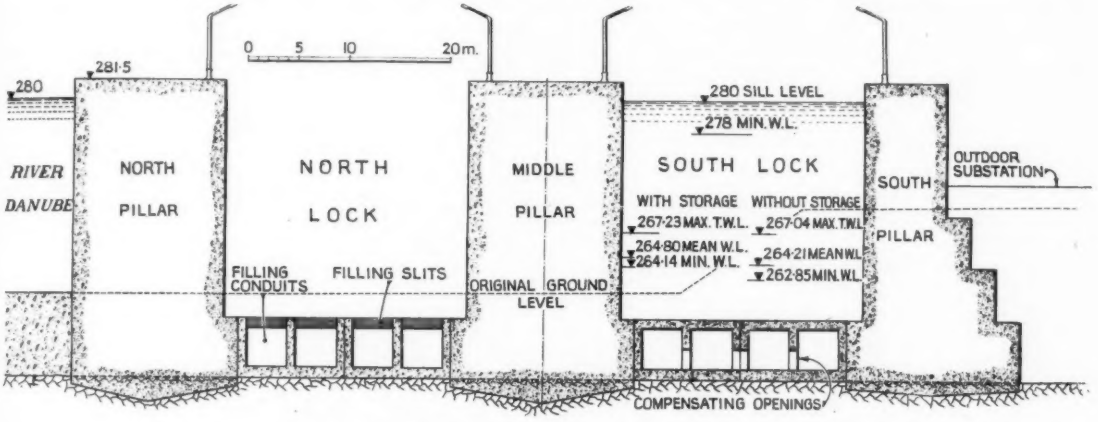


Fig. 3. Transverse section through the locks of the proposed Aschach station

An inspection passage will be built under the spillway sill. To resist the impact of the bed load the spillway pillars and foundations will be faced with granite.

Power House

The power house will occupy the space between the lock structure and the spillway, being separated from the latter by a pillar 12 m. wide. The machine hall will be 166 m. long, 16 m. wide, and about 12 m. high above the floor, which will be at exciter level. The rails for the two 180 ton portal cranes already mentioned will be laid at a gauge of 18 m. along the tops of the machine-hall walls, and a series of openings, closed by temporary covers, will be left in the machine-hall roof to enable heavy parts to be handled by the portal cranes. For lighter duties a 30 ton gantry crane will be provided within the machine hall.

Detailed investigations, covering the sequence of storage operations throughout the entire chain of Danube power stations, pointed to designing the generating equipment on the basis of a flow of 1,800 cu. m. per sec. during 91 days in a normal year and of a guaranteed cavitation-free flow of 2,030 cu. m. per sec. These studies showed that the optimum equipment would consist of four main generating sets, each consisting of a Kaplan turbine developing 84,300 h.p. at a flow of 450 cu. m. per sec. and 93,000 h.p. at maximum flow, coupled to a 72 MW generator, yielding 1,609 million kWh in an average year. In addition, a 4,000 kW auxiliary set will be provided, and auxiliary services will be further safeguarded by connection with the local 25 kV network.

In designing the main generating sets the novel

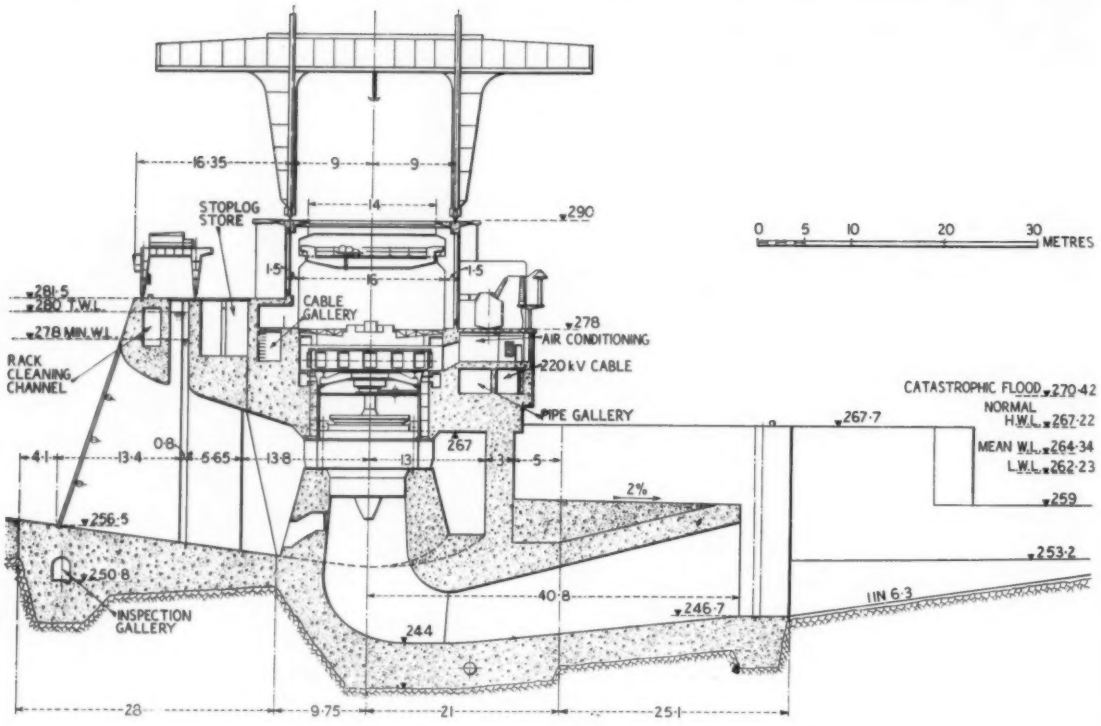


Fig. 4. Transverse section through Aschach power house

procedure will be adopted of arranging adjacent units to rotate in opposite directions. This will enable all the controls for each pair of machines to be grouped together on a common platform.

The alternators will generate at 15 kV and each will be directly connected to its own transformer, located on the platform above the draft tubes, which will step up to 220 kV. Oil-filled cables will convey the power to an outdoor substation on the right bank, from which six transmission lines will eventually be taken off.

The control room will be located in the administrative building to be erected on the right bank.

Implementation of Construction Work

The bridges of the standard-gauge branch line which connects Aschach with the main Vienna-Linz-Passau railway will have to be reinforced and the line itself extended a further 3 km. to the construction site. The branch road to Aschach will have to be enlarged and reinforced and on the left river bank a construction road will have to be built.

The cofferdams will have to be higher than at Jochenstein and Ybbs-Persenbeug. The bedrock in the vicinity lies at a level of 253 m. and is thus 8 m. below the bed of the river and 14 m. below the embankment.

The piling of the construction pits will be designed on the assumption that the water flow will be 6,200 cu. m. per sec., i.e., that the volume will not exceed the maximum experienced during the last 20 years. The piling will be erected to a height of 18 m. above the bedrock, i.e., to 271 m. Four construction pits are envisaged.

The main plant will be built in two construction stages; the first will include the spillway, separating pillars and southern lock, while the second will comprise the entire generating station and northern lock. At first, provision will be made for coffering off the site of the southern lock on the right bank and for two spillway openings on the left bank. At the same time compressed air will be available for founding the pillar near the left bank by the caisson method. Protected by this coffering work the southern lock and the oblique leader walls will be erected. Simultaneously the embankment walls in the head and tailbays will be built and the construction of the power house and other buildings initiated. On the left bank two spillway openings, excluding the sills, two pillars, and the spillway abutment will be constructed. In the river 15 caissons, 12 m. × 24 m., will be sunk for the foundations of the separating pillar.

In the next stage spillway openings 1 to 3 will be built, the sills being concreted at first only up to 261 m.

In the meanwhile the southern lock will have been completed and will be available for navigation. Consequently the construction pit pilings for the generating station and the northern lock can be erected. After completion of the underground structures for the turbines it will be possible to remove the cofferdams, to store the water up to 276 m. and to put the first turbine into operation, the gates being temporarily mounted 4 m. low on the uncompleted spillway sills.

As soon as the northern lock becomes usable, the southern lock can be laid dry and completed. During partial storage the filling and emptying of both lock chambers will be effected through the floor, and after

partial storage, the spillway sills will be concreted under the protection of the stoplogs.

This construction scheme will ensure that power generation is commenced with one generator after four years, and all four generators should be running after four and a half years.

Preparatory Work

Elaboration of the project and of the plan of execution was preceded by thorough preparatory work which it is impossible to describe in detail, and this will therefore only be summarised here. For opening up the building site, 97 vertical and two horizontal borings were made in the area of the main constructional work. For the first time an optical drill-hole probe was introduced to examine the system of inclined rock fractures. Evaluation of the vertical borings enabled the prediction to be made that during construction no serious foundation difficulties would arise. The horizontal borings will show whether sealing aprons will be necessary and whether the foundation can be grouted.

Hydrological conditions prevailing at Aschach were closely investigated. The plant has been designed in such a way that it would be capable of utilising 94·6% of the average flow for the period 1894–1949. The key curves, water levels and water volumes have been established both for the generating station and for the storage area. As is known, flood flows of 9,200 cu. m. per sec. were experienced 200 years ago and occurred again in 1954, and model tests to establish the weir dimensions were therefore based on a flood of 10,000 cu. m. per sec., which occurred 500 years ago. Preparatory work also included the establishment of the tailwater levels in order to have hydrological data available for the subsequent development of the next Danube plant downstream.

All relative phenomena in the storage area were closely studied by means of model tests. The normal floods occurring regularly every year leave the lower part of the storage area unaffected and make themselves felt only in its upper part, where the river embankments will be built up so as to keep flood risks to a minimum.

The preparatory investigations have already shown the advantages accruing to shipping from the construction of the Aschach plant. In the storage area all shallows and obstructions that now exist will be submerged and the full width of the river will be available for navigation. Even at the lowest levels two-way navigation will be possible, so it will be possible to abandon the existing signalling systems for the one-way sections. No fish passes will be necessary as the typical migratory species do not live in the Danube. On the contrary the storage basin will provide improved feeding grounds for the existing fish.

Energy Output

As already stated, the average utilisable power output will be 1,609 million kWh per annum, or 1,539 million kWh when reductions caused by the Partenstein, Kramesau and Jochenstein stations are taken into account. The energy output will amount to 1,309 million kWh in a dry year and 1,879 million kWh in a wet year.

Because of its relatively high net head and considerable storage volume, the Aschach plant is to function as the main power station for continuous storage operations of a future chain of plants on the Danube. Lowering the water level by one metre results in releasing 9·5 million cu. m. If this volume of water is released in addition to the natural flow of 1,511 cu. m. per sec., the generating capacity could be raised by 46 MW from 208 MW to 254 MW. By retaining 150 cu. m. per sec. through 18 hours the storage basin is filled again.

Architectural Features

A large construction such as a hydro-electric development must be built so as to fit harmoniously into the surrounding landscape. The engineers responsible for designing the Aschach plant have had continuous advice from architectural experts in order to ensure that a satisfactory design is achieved.

Power Construction Limited

A new consortium of established British companies —Power Construction Limited—has been formed to design, manufacture, deliver and construct power stations in any part of the world. The members of the consortium are: The Armfield Hydraulic Engineering Co. Ltd., an account of whose present activities appears elsewhere in this issue, Belliss & Morcom Limited, Cory Brothers & Co. Ltd. (a member of the Powell Duffryn Group, Lancashire Dynamo Holdings Ltd., and the Mitchell Engineering Group Ltd.

The range of experience and the facilities of its members enables the consortium to offer a comprehensive and completely integrated service embracing all stages from the planning to the commissioning of power stations, whether they be driven by steam, internal-combustion, or hydraulic prime movers.

Through the combined organisations of its members the consortium has representation in most parts of the world and offers generating sets and compressors in the following ranges.

Turbo alternators	Up to 15,000 kW
Steam engine-driven alternators	Up to 900 h.p.
Diesel engine-driven alternators	120–2,000 h.p.
Duel-fuel engine-driven alternators	120–2,000 h.p.
Pelton wheels	Up to 5,000 h.p.
Francis turbines	Up to 5,000 h.p.
Propeller turbines	Up to 1,000 h.p.
Kaplan turbines	Up to 500 h.p.
Air and gas compressors	Up to 12,000 c.f.m. at 120 lb. per sq. in. (lower capacities up to 7,000 lb. per sq. in.

In addition to the generating sets themselves the consortium offers advice on the selection of the most suitable type of power station; the preparation of specifications when required; site investigation; inspection and progressing during manufacture; shipping and insurance services; erection on site, including civil works; acceptance testing.

The address of the consortium is Power Construction Limited, 19, Berkeley Street, London, W.1.

The IAHR Montreal Congress

A series reporting the proceedings of the International Association for Hydraulic Research Eighth Congress held in Montreal, Canada, in August, 1959. Many of the papers reviewed are likely to be of particular interest to hydro-electric engineers and to the designers of hydraulic structures

PART TWO

SUMMARY ON AIR ENTRAINMENT, AIR SHAFTS AND VALVES

Aeration, aerated flow and deaeration processes are complex phenomena which are not yet open to rigorous mathematical and scale model analysis. They are important because they may cause deterioration and failures, especially in pressure type structures.

Fundamentals

Chanishvili (Paper No. 8D., U.S.S.R.) described investigations on vertical downward flow induced by a jet and phase interaction phenomena producing varying air bubble velocities according to the degree of aeration and depth.

Raymond and Simonin (Paper No. 6D., France) analysed air entrainment, bubble radius and pressure created by a cylindrical jet piercing a water surface with experimental evidence to show that subdivided energy of molecular origin appears as a swarm of bubbles.

Homma and Sakamoto (Paper No. 2D., Japan), on the other hand, photographed the shape and break-up of a vertical circular jet and measured velocity fluctuations with a pressure pick-up. This revealed that acceleration roughens the jet surface with gradual disintegration, air entrainment and retardation. They also endeavoured to analyse air density, water viscosity and surface tension effects.

Only one paper considered vortex formation *(Haindl No. 16D., Czechoslovakia)* from fundamental principles. This was supported by experimental data for circular tank floor orifices defining entraining vortex strength as a function of outflow velocity, circulation impulse and water depth. Important conclusions are drawn for entraining vortices:—

(i) Froude's law is not valid.
(ii) For similar circulation impulses the relative origin heights are equal for the same Reynold's number.
(iii) The air quantity depends on their moment of momentum.

Hydraulic Structures

SECONDARY INTAKES FOR PRESSURE SHAFTS

There has been much experience in France with secondary intakes and *Cotillon (Paper No. 17D.)* classified and illustrated alternative designs under the headings of:—

(i) Natural deaeration.
(ii) Deaeration under pressure.
(iii) Air entrainment suppression.

the principles of which are well known.

Paper No. 10D. (Plichin and Sevin, France) described the "Casserole" deaerator utilising the shock principle to limit the emulsion speed and penetration with an energy dissipator where the downward velocity is less than that at which bubbles rise to the surface. This is simply a flat bottom cylinder with concentric rings below the vertical pipe. The leading dimensions, based on model tests, are given.

Boughton (Paper No. 5D., Australia) described a more complex air exclusion system comprising a siphon battery and vortex chamber to divert 400 cusecs into a tunnel 400 ft. below. Interesting test techniques were the use of kerosene to simulate prototype air bubbles and high speed photography of wooden floats for measuring vortex chamber velocity components on which inlet and outlet shapes were based. Axially symmetrical vortex chamber flow eliminates the air carried down during siphon priming and depriming as bubbles which ascend the shaft.

AIRSHAFTS

Gate and valve performance is allied to the associated airshaft design and *Paper No. 11D. (Le Mehauté, Canada)* discussed air entrainment by hydraulic jump in a gallery and air demand as a function of gate opening and shaft dimensions to give the appropriate pressures for different modes of flow. Experiments seem to confirm this theoretical treatment which warrants additional prototype observations. It might be correlated with *Mura, Ijuin and Nakagawa's* limited field data *(Paper No. 12D., Japan)* which suggests that air demand depends on the aerated water/critical area ratio, gate, conduit and outlet details. The relative velocities between entrained air and water flow are considerable and measurements including air concentration by electrical or gamma radiation methods are envisaged for future investigations.

Neither paper shows agreement with Kalinske and Robertson's curve, which indicates that apart from Froude number, downstream head loss and gallery submergence are important airshaft design factors.

SPILLWAY CONDUITS

Paper No. 4D. (Martins, Portugal) confirmed that circular spillways serving long free-flow tunnels demand adequate aeration to ensure freedom from vibration, cavitation and pressure surges which may induce inlet vortices with consequent discharge limitations. The paper examined the effect of the elbow/shaft location relative to air entrainment, airshaft

outlet submergence, ventilation devices and deflectors based on model tests.

Downstream air shafts are dimensioned to the U.S. Corps of Engineers data for tunnel gates relating Froude number and the Kalinske/Robertson and Campbell/Guyton curves.

Paper No. 1D. (Okada, Kudo and Fukuhara, Japan) concerned field measurement of high velocity flow and air content in a closed conduit to ascertain air bulking. Three Pitot tubes were embedded in the unpressurised free flowing spillway to check a 1:15 model. Although actual and computed velocities agreed, the air entrained in water samples gave scattered measurements below those calculated by Hull's formula. The discrepancy, attributed to non-uniform aerated water density, is confirmed by correlating Froude's number with water/Pitot depth.

SPILLWAYS

Sananes (Paper No. 13D., France) reported on an overflowing semi-circular crest where the water clears the downstream vertical wall if pressure is maintained below the sheet but otherwise air is entrained and unstable flow results. The air demand, where no defined surface exists between the gaseous and liquid phases has, so far, resisted analysis and model tests for air/water mixtures do not obey similitude laws.

Vertical face models with variable downstream slopes and controlled aeration were used to deduce under-nappe air entrainment from evacuation times and to reveal surface and air friction effects as well as geometric parameters such as channel width, crest radius and height which influence the volume under the sheet at atmospheric pressure and where the height of a dead water zone depends on the crest head. Model measurements agree with calculations.

Paper No. 9D. (Cebertowicz and Jarocki, Poland) concerned combined spillway sector gate and ground sluice operation where under certain conditions:—

(i) A hydraulic roller occurred decreasing sluice area.
(ii) The external/internal surface pressure difference periodically broke and reformed the thin water layer

accompanied by negative pressures, air removal, shock, noise and vibration. Although some experimental devices were partially successful, the simplest solution was to aerate the underpressure space, but simultaneous operation is better avoided.

Other Structures

SURGE CHAMBER WITH AIR CUSHIONS

Paper No. 14D. (Grcic, Jugoslavia) presented a mathematical and graphical analysis showing the influence of air volume on the oscillation and stability of closed and vented surge chambers, including those fitted with synchronised regulating valves. The conclusions confirm similar investigations elsewhere.

PNEUMATIC BREAKWATERS AND ICE ELIMINATORS

Paper No. 7D. (Bains and Hamilton, Canada) discussed water flow induced by rising bubbles and described experiments with air injection through tank floor orifices. The jets above the air source and the horizontal surface flows display turbulent velocity distributions. The theoretical analysis couples water jet energy, discharge and momentum with the air discharge rate and orifice size. This leads to a Q_w/Q_a equation exponents which might permit application of scaling laws to bubbler system characteristics.

HYDRAULIC AIR COMPRESSORS

Faure and Pugnet (Paper No. 15, France) discussed an experimental flow study with entrained air. The principles and theory outlined concerned emulsion formation, water mixture transport, deaeration, losses and efficiency.

Papers (Nos. 18D and 20D. by Bonnin and Lamble, France) examined the theoretical and experimental conditions under which air/water emulsions remain stable and homogeneous, the phenomena associated with heterogeneous emulsions and the fundamentals of highly concentrated gaseous/liquid movement. Many problems concerning economic vertical transport velocities, losses and emulsifier design await solution before a stable and efficient industrial compressor for high air concentrations can be constructed. The experiments are, however, encouraging enough to justify large scale investigations.

SUMMARY ON NAVIGATION LOCKS

The importance of high lift navigation locks in multiple purpose projects was emphasised by papers which dealt with practical problems including model testing and supporting field observations.

Generally these related to longitudinal culvert filling and emptying systems, or variants of it, and not unexpectedly, they disclosed common design and operational problems. A paper by *Nelson (No. 11B., U.S.A.)* on "Hydraulics of Lock Filling and Emptying Systems" constituted an excellent basis for comparison with practice elsewhere which was outlined in:—

Paper No. 19B., Lejeune (France). "High Head Locks Recently Constructed in France."

Paper No. 9B., Khalturin, Lyubimov, Mikailhof and Onipchenko (U.S.S.R.). "Hydraulic Investigations of Navigation Locks in Volga River Basin."

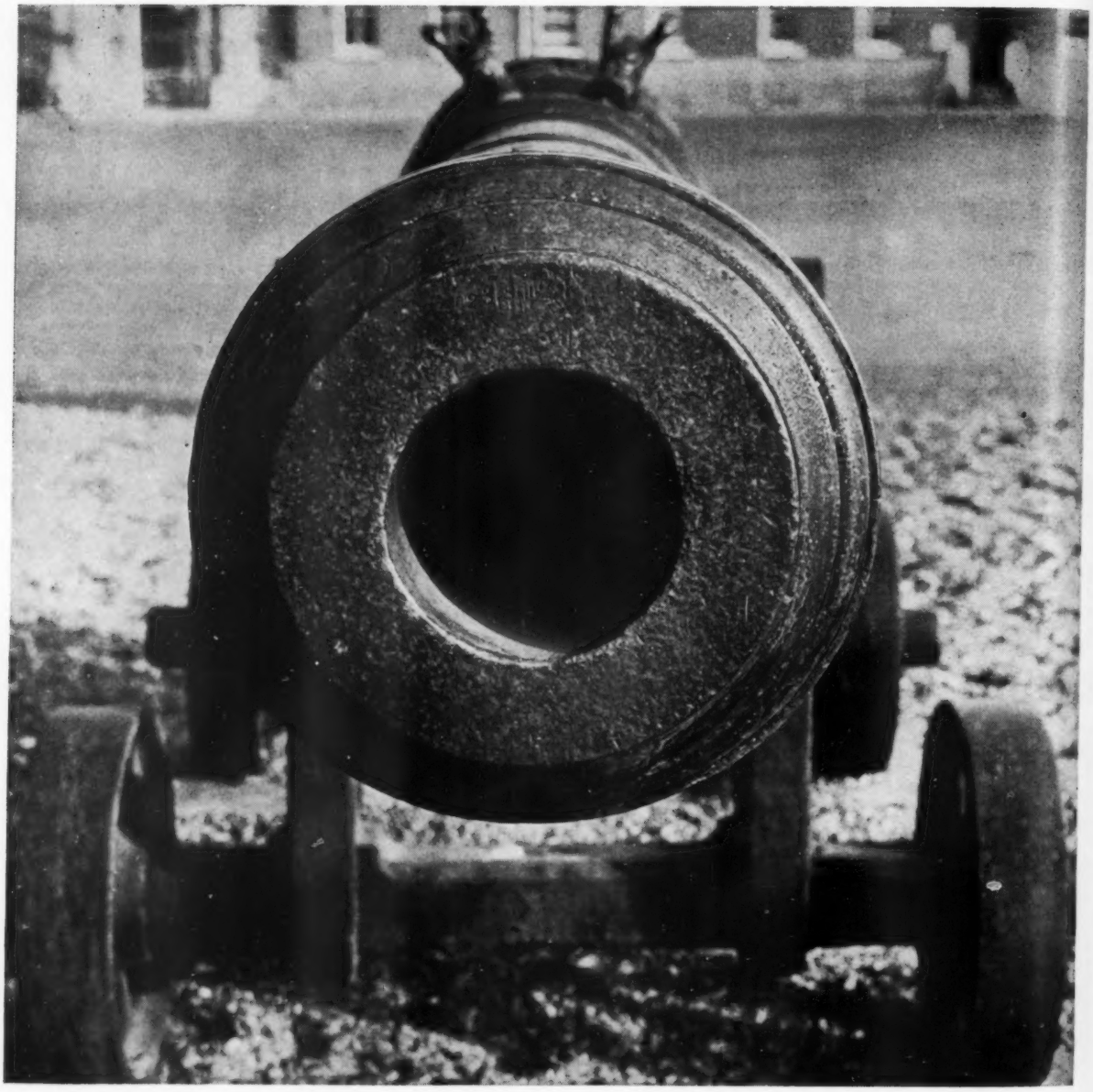
Paper No. 16B., Grengg and Wehrshutz (Austria). "Hydraulic Lock System at Aschach Power Station."

Paper No. 15B., Pariset and Galiana (Canada). Carillon Lock and specialised contributions devoted to detail problems.

Lock Approaches

Unless water is uniformly withdrawn from and returned to the approaches, cross currents, eddies and vortices will reduce the filling and emptying system efficiency and may endanger vessels moored in the upper and lower pools. *Partenscky's* theoretical study *(Paper No. 4B., Germany/U.S.A.),* however, suggests that negative translational waves do not seriously affect vessel manoeuvrability, acceleration/deceleration and peak tow-rope forces. This is supported by prototype strain gauge measurements, and calculations including ship and barge tow resistance, are extended to determine the upper pool dimensions or the lock filling rate. Double locks were also mentioned in *Paper No. 5B. (Canisius and Jambor, Germany)* which covered the structural aspects of head filling systems with vertical lifting gates and energy dissipating devices and their use for flood discharge.

Nelson and Johnson (Paper No. 12B., U.S.A.) discussed lock intake vortex problems. The risks to small craft and personnel and those associated with



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the entrained air release and structural damage by clogging with ice, logs and debris were illustrated by actual cases under model and prototype conditions. The erratic nature of vortex phenomena is clear since they occur irregularly in time and magnitude during a filling cycle and may persist until culvert flow reverses. The paper appeals for analysis of this problem and while conceding that reduced vorticity in the model improves prototype conditions admits that this is not necessarily to scale.

Intake Manifolds

All sections should carry equal gradually accelerated flows merging into those from preceding ports without excessive head loss or shock. The entrance velocities should be low enough to prevent vortices, air entrainment and other conditions which produce undesirable lock chamber disturbances.

Lock Valves

Sector valves of streamlined structural design and robust construction with the trunnions located so that the curved skinplate faces downstream to avoid air entrainment, excessive submergence and negative pressures causing cavitation damage, vibration and noise during opening, are now almost universal. Valve operation problems were treated in:—

Paper No. 13B. (Nelson, Johnson and Fidelman, U.S.A.). The low pressures created downstream during the initial valve opening period cannot safely utilise air venting. Model tests showed, however, that a water feedback to this critical zone is a promising corrective and reduces the valve dynamic loading. Field confirmation is awaited.

Paper No. 3B. (Shu Tien Li, U.S.A.) analysed high constant speed and differential head actuated, variable speed, lock chamber volume, valve opening and flow, control systems for which lock turbulence factor and longitudinal dynamic disturbance concepts are introduced to derive differential equations which can be solved numerically on a computer.

Paper No. 17B. (Ransford, France) drew attention to waterhammer risks in filling systems from valve failure or mal-operation causing instantaneous closure, thus creating cavitation pockets and possibly pressure inversion. Valves or shafts may partially protect the complicated waterway system but may be unsatisfactory at low speeds because coalesced air bubbles in the culverts may escape violently into the lock chamber. The limitations of an approximate mathematical analysis and possible remedies are examined.

Lock Culverts and Manifolds

The staggered gradually expanding wall ports are streamlined to minimise head loss and boundary flow separation and to provide internal jet diffusion, partial velocity head recovery with reduced exit speed and turbulence.

Nelson's curve relating rate of surface rise with acceptable turbulence to water cushion depth offers an interesting criterion for selection of wall port or bottom lateral systems. Surging is a minimum when these facilities are located in the middle third of the lock except for unusually long chambers, where small random moored tows may warrant split lateral systems in the middle thirds of the upstream and downstream halves. Where practicable, uniform port discharge is ensured by tapered culverts. Detailed aspects were considered in:—

Paper No. 6B. (Feir, Canada) which examined "Lock Culvert Systems Considering Non-Linear Friction" and compared model with calculated behaviour for instantaneous valve opening, assuming the upstream port system reduced to a single port. Although qualified agreement between filling curves and lock chamber motion prevailed over the first oscillation, such an analogue has accuracy limitations.

Paper No. 18B. (Escande, France) which provided a step-by-step procedure for calculating port system flows under semi-permanent conditions for a given head between the reservoir and lock chamber.

Lock Filling/Emptying Process and Hawser Forces

Model tests and experience prove that, for lifts exceeding 10–15 m., the valve-controlled longitudinal culvert distribution system feeding a series of bottom lateral or side wall orifices, permits rapid filling and emptying with minimum undesirable effects on passing boats. The design problems are:—

(i) Dimensions and shape of filling and emptying culverts and the associated port system.
(ii) Control valve sizes and their operation for:—
  (a) Minimum filling and emptying times.
  (b) Culvert flow rates low enough to prevent harmful agitation in the lock.

Paper No. 7B. (Noseda, Italy) presented a generalised theoretical and confirmatory experimental analysis, embracing the valve timing laws and other factors defining the minimum filling time, rate of lock level rise and other characteristics which determine the appropriate opening without exceeding a pre-established flow.

Paper No. 22B. (Labetoulle, France) and *Paper No. 20B. (Peutot, France)* examined the influence of various parameters on mooring forces caused by translational wave propagation and outlined a Craya/Schnyder Bergeron graphical method for their determination throughout lockage allowing for ship and lock floor discontinuities.

Special Designs

New ideas or novel designs were introduced in:—

Paper No. 2B. (Grzywienski, Austria) which illustrated several non-approach channel, transverse lock wall and floor filling designs which were model tested for the Danube.

Paper No. 8B. (Vasiliev, U.S.S.R.) which derived approximate linearised differential equations for water level fluctuations, ship forces and lockage conditions, for general application but with particular reference to inclined ship elevators during acceleration or sudden stoppage.

Paper No. 1B. (Jarocki, Poland) which outlined the principles of the high lift pneumatic lock for sites where water economy is essential. It comprises two compressed air chambers, each containing a tank full of water, one of which rises and the other falls when the air valve is opened. Analytical and laboratory investigations on:—

(a) Tank elevating air pressure.
(b) Tank motion, time and braking.
(c) Sealing against compressed air and water leakage losses

generally agree. The proposal compares favourably with hydraulic locks, but design and construction

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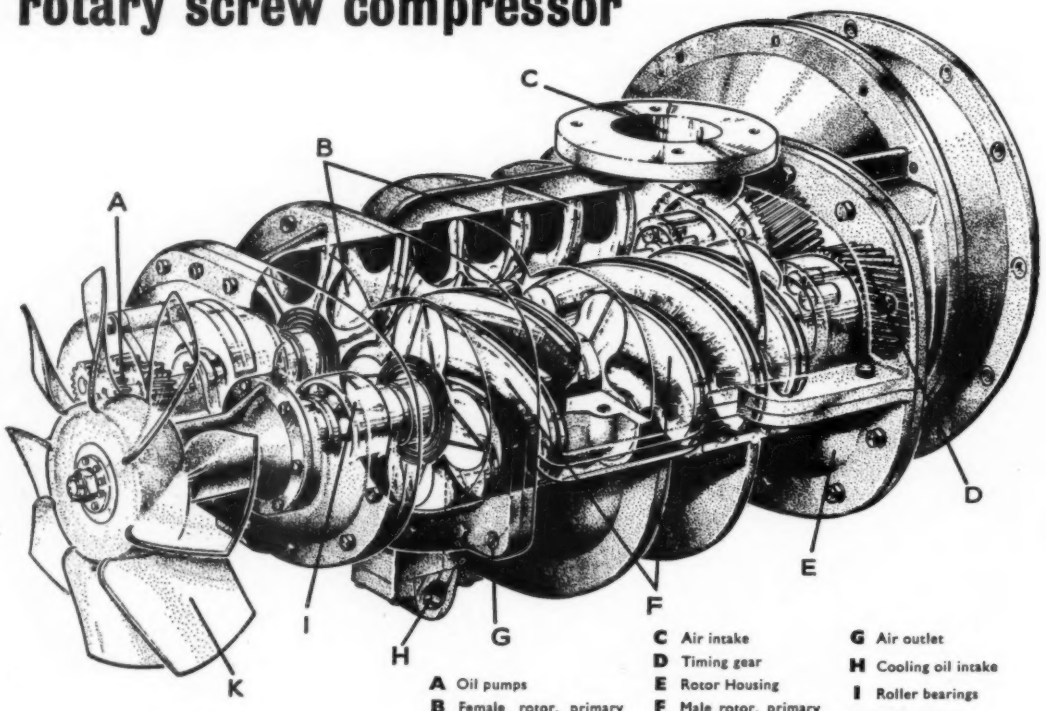

A Oil pumps
B Female rotor, primary and secondary stage
C Air intake
D Timing gear
E Rotor Housing
F Male rotor, primary and secondary stage
G Air outlet
H Cooling oil intake
I Roller bearings
K Oil-cooler fan

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difficulties seem likely.

Paper No. 14B. (Petrikat, Germany) which gave constructional details and characteristic curves for a rotating hook type flap gate for head filling over the lock width. In one installation, vibration appeared:—

(i) At initial opening with a frequency of 10 c.p.s. This was eliminated by modified rubber seals.

(ii) 4·5 minutes after opening with a frequency of 4·5 c.p.s. This was of short duration and was traced to back surging producing a pressure rise and air/water mixture compression in the antechamber. Compensating pressure relief openings between the antechamber and lock aided by gate damping diminished water load changes and vibration build-up.

SUMMARY ON DENSITY CURRENTS

Reservoir Silting

Paper No. 8C. (Levi, U.S.S.R.) provided a reservoir silting theory based on non-uniform unsteady underflow originating from alluvium transport. The mathematical treatment is founded on differential equations embracing incipient instability of underflow interface, upper boundary velocity gradient and minimum turbidity and derives formulae in terms of:—

(i) Sediment concentration.

(ii) Steady regime velocity.

(iii) Flood wave propagation velocity.

(iv) Rise of underflow level in flood storage process which quantitatively determine reservoir silting characteristics and duration of silt discharge.

Note No. 12SII. (Jordaan, South Africa) also dealt with silt transport through reservoirs by induced density currents.

Reduction of reservoir siltation by abstracting heavily charged water has practical applications and *Paper No. 10C. (Harleman, Morgan and Purple, U.S.A.)* presented an analytical approach to selective withdrawal from a vertically stratified fluid based on experiments with floor outlets discharging a lower heavier layer until a critical draw-down point is reached. This is expressed in dimensionless variables defining fluid and geometric conditions and might be applicable to circulating water intakes where temperature density differences are involved. In this connection, *Barr (Great Britain)* contributed *Paper No. 6C.* on "Observations of Small Scale Density Currents," which dealt with heat dissipation model design and operation.

These formed a useful approach to *Paper No. 3C. (Goda, Japan)* which reported density current observations in an impounding reservoir as indicated by the hydraulic, physical, chemical and geological conditions with which they are associated.

Channels

Papers No. 12C. (Bata, Jugoslavia) and *No. 14C. (Bonnefille and Goddet, France)* were devoted to mathematical analysis of density current flow mechanisms in long channels with respect to bed and interface frictional resistances, velocity and shear stress between a dense layer moving below a light one and correlation of experimental observations with velocity distribution, suspended load concentration and temperature.

Estuarine Conditions

Japanese rivers discharging fresh water into non-tidal seas exhibit horizontal and vertical density gradients with moderate mixing. Three papers were devoted to salt water wedge intrusion.

Hamada (Paper No. 2C.) reviewed stratified density currents theoretically and concluded with some experimental evidence showing that the steady internal jump associated with wave energy transmission losses and surface flow Froude number are controlling factors.

Paper No. 5C. (Akikusa, Sato, Kikkawa, Kishi and Tominaga) examined salinity distribution and deduced that the duration of exchange flow was longer for non-uniform than for uniform gradients.

Paper 4C. (Otsubo and Fukushima) reported on field observations of river mouth penetration in terms of fresh water entrainment and salinity concentration from which the salt wedge shape, shear stress, critical wash-away discharge, turbulent diffusivity and decay are determined.

Allen and Price (Paper No. 16C., Great Britain) described siltation in Thames and Mersey docks and tidal basins where density currents arose from salinity differences between two interconnected or periodically separated bodies of water. Preliminary experiments confirmed the underlying mechanism and investigations for quantitative comparisons between theory and field measurements have been initiated.

Other Allied Topics

The remaining papers are listed to complete the record.

9C. Gustafsson (Sweden):
Surface Turbidity in the Indalsalven River associated with a Dam at Bergeforsen.

11C. Kivisild (Canada):
Density Currents Effects in Deas Island Tunnel Sinking Operations.

15C. Abraham and Diephuis (Netherlands):
Clay Suspension as a Substitute for Sea Water in Model Tests on Density Currents.

7C. Schlag (Belgium):
Flow of Two Superposed Fluids of Different Densities over a Weir.

13C. Bata (Jugoslavia):
Propagation of a Gasoline Wave through Water in an Inclined Circular Annulus.

1C. Leeuw (Israel):
Dead Sea Density Currents.

(To be continued)

Safety in Civil Engineering. Over 15,000 copies of the "Safety Guide for Works of Civil Engineering Construction" have been sold since it was first published by the Federation of Civil Engineering Contractors last March, and a reprint has been necessary. Copies of this pocket-size guide are available from the Federation at Romney House, Tufton Street, Westminster, S.W.1, price 1s. per copy.

As a further step in its safety campaign, the Federation has set up a Civil Engineering Safety Training Course for Supervisors. The course lasts two days and covers such subjects as causes of accidents on construction work, site tidiness, excavations and earthworks, staging, electricity, lifting and carrying and the cost of accidents. Three films will be shown, and at the conclusion of the course there will be an opportunity for a general discussion and questions.

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Weighing only 3 kilos (6.6 lb.), it is the lightest self-rotating pneumatic plug-hole drill in the world. Twin-controls enable the Wasp to be used with either hand—a decided advantage when working in restricted and awkward positions, both common problems in the building industry.

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The Wasp is suitable for use with Sandvik Coromant plug-hole drills with tungsten carbide inserts, and is supplied complete with accessories and spares in a strong transport case. It is the latest addition to the many pneumatic tools in the Atlas Copco range. Represented in ninety countries, Atlas Copco is the largest organisation in the world specialising solely in the design and manufacture of compressed air equipment.

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P.34

Open day at Armfield's

Fig. 1. One of the 1,600 b.h.p. Francis turbines for the Fort Rosebery power station

THE uses and potentialities of "micro" water-power generating sets have been discussed at length in several past issues of WATER POWER. The fact that these micro plants do not require extensive civil engineering, are easy to install and need little maintenance, made them popular for power production on farms, little villages and light industry where a stream and a modest head were available. One of the pioneering companies in the field of small to medium water-power equipment is the Armfield Hydraulic Engineering Co. Ltd., Ringwood, Hants., which was founded in 1875 and whom we had the opportunity of visiting recently.

On the home market, the use of these micro plants by light industries, such as paper and flour mills, is becoming limited due to the amalgamation of the smaller factories and the tendency to mass produce in giant concerns needing far more power than can be obtained from small sets. But in other areas of the world there is a growing interest in small and medium-size power stations, especially in highland communities with small local industries. The United Nations report* has suggested that countries like Turkey, Greece and Yugoslavia would benefit by the installation of such isolated plants due to the abundance of small streams having heads ranging from 10 to 60 ft. and with a compactly grouped population in small towns and villages. Similar conditions are also found in the Indian sub-continent. This fact has encouraged the Armfield Hydraulic Engineering Co. Ltd. to join a consortium with the Power Construction Co. Ltd. which includes Lancashire Dynamo Group Sales, Belliss & Morcom, Mitchell Engineering Group, and Cory Brothers and Company, primarily to supply complete generating stations.

But the uses of small hydraulic equipment do not stop there. Small sets for demonstration and educational purposes are in great demand due to the recent emphasis on technical education and this is a speciality of the Armfield Hydraulic Engineering Co. Ltd. A variety of laboratory test equipment is being supplied to many colleges in this country and in Commonwealth countries which include Pelton wheels, Francis turbines, propeller turbines, Kaplan turbines and axial-flow pumps. These laboratory machines have been standardised so that a high efficiency for these models is obtained. The units, which vary in power between 2 and 6·5 b.h.p., are supplied in complete and compact sets together with their own measuring tanks with V-notches, dynamometers and governors as may be appropriate.

The sizes of the turbines already manufactured by the firm vary from 2 b.h.p. to 1,600 b.h.p. and plans for machines of 5,000 b.h.p. and over are being studied. This is indeed a very good rate of expansion,

* "Rural Electrification," Vol. I (E/ECE/173). Published by the United Nations Economic Council and obtainable from the Sales Section, European Office of the United Nations, Palais des Nations, Geneva, Switzerland. Price 11s. or $1·50.

Runner of prosperity

Recently another lasting contribution to technical progress and prosperity was made by NOHAB quality products. The Stornorrfors Power Station, and with it the large Francis turbine at left (three NOHAB turbines are installed there) was inaugurated on September 13, 1959.

Ample supply of energy in convenient forms is a fundamental condition for our standards of living. NOHAB turbines help to produce electric power in water power plants all over the world.

Below is a table of recent important orders for NOHAB turbines.

Specifications of large water turbines already delivered, or under delivery by NOHAB

Power Station	Number and type of turbine	Head		Output Metric H.P. per turbine	Speed r.p.m.	Comments
		ft.	m.			
STORNORRFORS Sweden In service 1959	3 Francis	246·0	75·0	200,000	125	The most powerful water turbines in the world in service.
LASELE Sweden In service 1956	2 Kaplan	170·6	52·0	87,200	150	This is the highest head for which a Kaplan has been installed in Sweden.
PORSI Sweden In service 1960	2 Kaplan	118·0	33·0	120,200	115	These will be the most powerful Kaplan turbines in Sweden.
MAEL Norway In service 1957	1 Francis	147·6	45·0	51,000	150	With regard to dimensions, the largest turbine in Norway.
PANCHET HILL India In service 1958	1 Kaplan	113·0	34·4	80,000	125	India's largest Kaplan, in output as well as dimensions.
RINCON DE BAYGORRIA Uruguay In service 1959	3 Kaplan	48·2	14·7	49,500	79	With regard to dimensions, South America's largest turbines.
FURNAS Brazil In service 1962	4 Francis	308·4	94·0	230,000	150	One of the most powerful water turbines in the world.

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since only 10 years ago the largest machine manufactured developed a mere 200 b.h.p. Under construction now are two turbines, each of 1 MW output at 600 r.p.m., for the Fort Rosebery power station which will operate under a fall of 100 ft. Water will be supplied by a 54 in. diameter pipeline of about 500 ft. length. The entire fabrication of the turbine casing is being carried out at the workshops at Ringwood and the turbine runner is cast in aluminium bronze and is fully machined and hand polished to ensure high efficiency. The interesting feature of the Fort Rosebery scheme is that the power station is below the maximum flood water level, and is therefore constructed to be watertight and to withstand flood waters of up to 25 ft. above floor level.

The design and calculation of the governors for the turbines are carried out at Armfield's and three types are on production, for the small technical college sets, for installations of 750 b.h.p. output and for 1,600 b.h.p. sets such as those at Fort Rosebery. These governors are usually of the oil-pressure type and utilise the servo principle where a small precision speed-measuring head is used to control a relay valve which regulates the supply of high-pressure oil to a piston and cylinder. The action of the piston regulates the turbine guide vanes. Mechanical feedback motions are incorporated in the valve gear for stable speed control under difficult site conditions, such as when the turbine is fed by a long pipeline. On the other hand, complete remote-control manipulations can be obtained by additional equipment.

A further expansion resulted in the construction of sluice gates. By 1960 a gate with a clear opening of 26 ft. by 16 ft. will be erected in Nigeria and others have been commissioned by river and drainage boards at home and abroad and by the North of Scotland Hydro-Electric Board and The Central Electricity Generating Board. A considerable amount of work has been effected in the design of automatic gates, and simple models have been devised for demonstration purposes. One model is in the form of a tainter gate with a counterbalance weight on the other side of the pivot. The gate is shut by the weight of a concrete block which rests in a channel separate from the main stream. Water flows into the side channel over a V-notch and discharges through an orifice. The principle involved is that the inward flow over the notch is a function of $H^{\frac{5}{2}}$, whereas the outward flow through the orifice is a function of $H^{\frac{1}{2}}$, so that any variation of head upstream will be magnified correspondingly in the side channel. This affects the buoyancy of the concrete and the gate is opened or shut automatically by the action of the counterbalance weight or the concrete block. Another type of model gate is the automatic tilting gate shown in Fig. 2. This is a flap-type gate hinged at the bottom of the flume. It is controlled by a linkage attached to the top of the gate and connected to a counterbalance weight. As the head of water over the gate rises, the flap opens, allowing an increased flow of water but maintaining the upstream head near constant. There are, of course, many other types of model gates under investigation, such as tipping gates, floating weir gates, float-operated tilting gates, etc., to enumerate a few.

Among the recent developments of the firm is an extremely mobile and versatile axial-flow pump designed primarily for flood irrigation and land drainage, but it can have other applications in industry or agriculture. Known as the Maxiflow pump, it can have a discharge of about 4 cusecs against a head of 18½ ft. In actual fact this pump consists of a propeller mounted inside a tube. The high efficiency obtained is due to the careful design of the propeller and the straightener blades which incorporate aerofoil characteristics. Its other feature is a 10 in. propeller diameter with speeds of between 1,350 and 1,750 r.p.m. using a 15 b.h.p. source. This makes it exceptionally handy for receiving power from most tractors via a step-up gearbox of 1 : 2·7 ratio which is built in all Maxiflow pumps. The pumped water is carried away by 8, 10 or 12 in. diameter flexible pipelines to wherever required. If electrical power is available, the Maxiflow pump can be supplied with a flange-mounted motor. These motors are available either for 50 cycles or 60 cycles and also totally enclosed or drip-proof. In some cases, where corrosion effects are particularly strong, the impeller and the straightener blades are made of special corrosion-resisting metals.

Fig. 2. Model of automatic tilting gate

Aluminium Replaces Steel Tubing

A saving of 30¢ a foot for a total of 33,000 ft. of pipe was achieved in the construction of the Williams Fork Dam near Parshall, Colorado, by using aluminium instead of steel tubing to dissipate the heat generated by the hardening of the concrete. In some instances this heat is sufficiently severe to crack the concrete—hence the use of the tubing for a water-cooling system. Mountain States Construction Company, builders of the dam, report that the savings resulted from the excellent coiling and bending properties of the aluminium tubing, together with the elimination of couplings and the reduction of points of possible leaks. Corrosion resistance gives this metal a life-long advantage, as it must be left in position and filled with cement when the concrete hardens.

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Abstracts from the World Technical Press

Surge-Tank Investigations

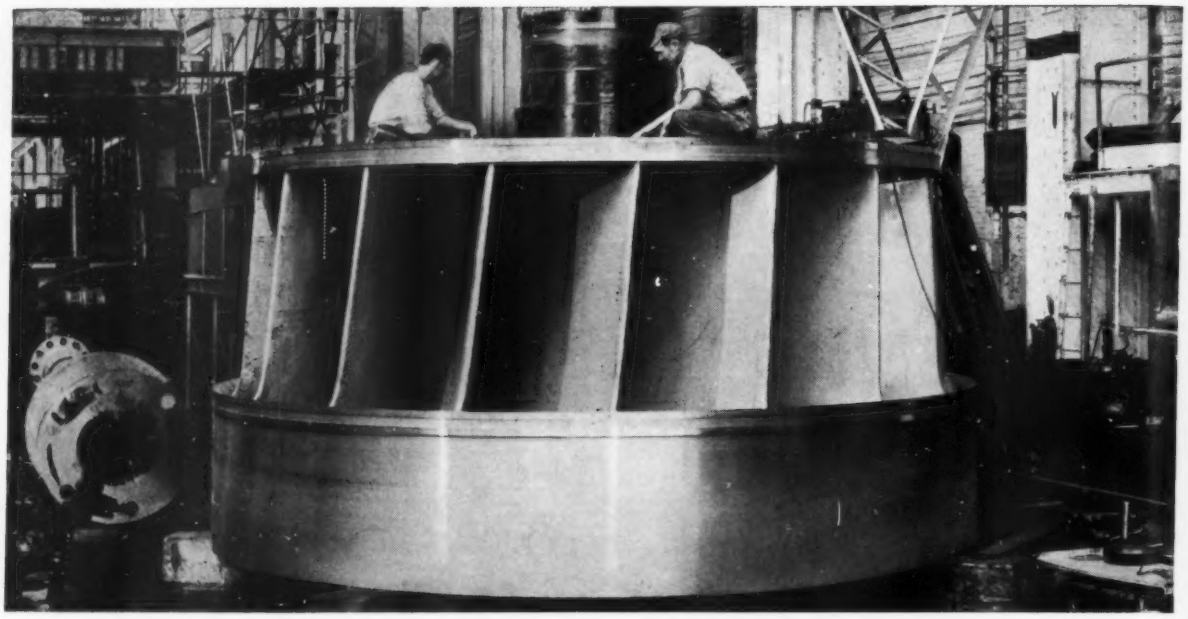
Proceeding from the general laws and rules of the established theory of surge-tanks, the author examines the possibility of applying a graphical method to the investigation of oscillations in a surge-tank fed by two conduits each connected with an independent reservoir. The figures illustrate the graphical determination of oscillations for various instances of instantaneous opening. Details are also given of the conditions to be complied with in order to ensure the validity of the hypothesis according to which the relation between the two conduits under steady flow also applies to unsteady flow. If these conditions are complied with, the computation can also be applied in exactly the same manner to the general case of a surge-tank fed from a single reservoir. (Peter Athanassof Kaichev, Civil Engineer, Sofia, *La Houille Blanche*, Vol. 14, No. 4, May-June 1959, p. 361, 5 pp., 6 ff.)

The Motec Balancing Reservoir

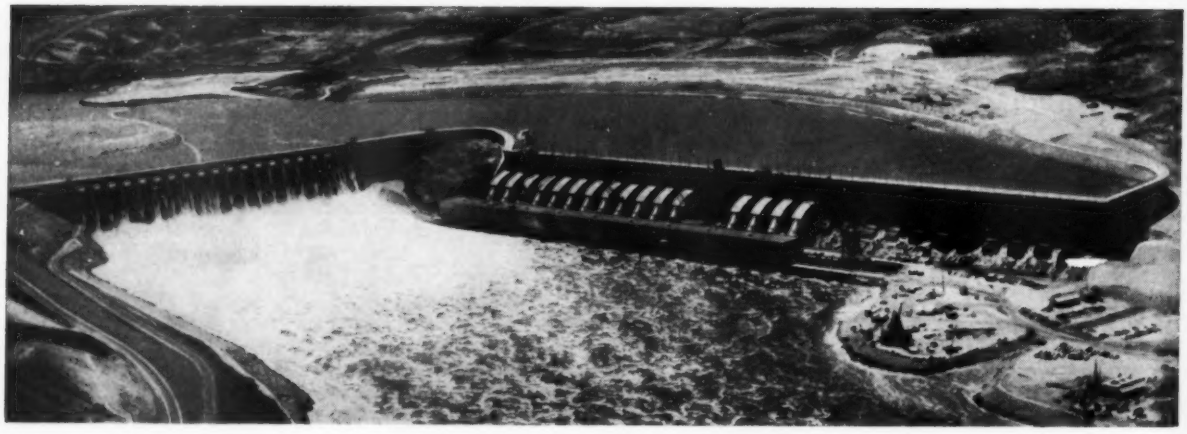
This reservoir is part of the installations of the Gougra River development; the river being a tributary of the Rhône through the Navisence River. This three stage development operates plants at Motec (69 MW), Vissoie (45 MW) and Chippis-Navisence (50 MW). The main purpose of the Motec balancing reservoir is to offset the wide fluctuation of the summer flow of the Navisence River so as to feed a regular power discharge to the Vissoie plant. Furthermore, its useful volume ensures three and a half hours of full-load running, not to mention at slack hours or week-end periods the pumping of the Navisence River into the Moiry Reservoir upstream. Altogether, its position between the first and second stages of the scheme gives much more flexibility to the running of the development as a whole. The project of the reservoir and construction work are described in detail. Following extensive borings and other preliminary work as well as a wide range of test models at the Hydraulics Laboratory of the Federal Technical University in Zurich, the layout selected is as follows. A fixed weir with a bottom outlet and a discharge capacity of 12 cu. m. per second conveys the Navisence waters through two parallel sandtraps into the 150,000 cu. m. balancing reservoir, about 300 m. long by 100 m. wide, with a depth varying between 7 and 11m. The Motec powerhouse stands on the left bank of the reservoir at the toe of a rock wall, while the Navisence diversion canal runs along the right bank. The old Motec-Zinal road which cut through the reservoir site, has been relocated along the right bank of the diversion canal. The reservoir involved a volume of excavation of 104,500 cu. m., while its banking required the placing of 88,500 cu. m. of earthfill. The impervious lining of the apron (14,000 sq. m.) and of the inner slopes of the new bank (12,000 sq. m.) is made of a bituminous concrete of the following composition per 1,000 kg.: 457 kg. of 0 to 3 mm. sand: 305 kg. of 3 to 8 mm. gravel: 162 kg. of 80/100 filler, and 76 kg. of Mexphalt bituminous binding material. The advantage of this bituminous concrete is that it can be placed without any joints, and follows closely any uneven settling of the bank without cracking. Its 6 cm. thick layer rests on a mattress of gravel consisting of a draining and filtering layer 12 and 10 cm. thick respectively, and was placed in two 3 cm. layers. To ensure a perfect binding between these two layers, which had to be placed at an interval of a few days, the bottom one was coated with a bituminous spread. Lining, which had to be suspended in rainy weather and at temperatures below 6°C, started in July 1957 and was completed in August 1958. The quality of the bituminous concrete was under permanent control in a laboratory installed on the construction site. The awkward problem of the jointing of the bituminous lining with concrete works was solved by providing a groove between the two, and filling it with bituminous cement. The imperviousness tests carried out during the impounding period in August 1958, gave excellent results. At full reservoir and a maximum depth of water of 11 m., total losses did not exceed 4 litres per second, corresponding to no more than 0·15 lit. per 1,000 sq. m. of wet area. The project of this balancing reservoir was worked out by Motor Columbus A.G., Baden, who were also responsible for the supervision of construction work. (Dipl. Ing. André Robert, *Bulletin Technique de la Suisse Romande*, Vol. 85, No. 18, August 29, 1959, p. 253, 8 pp., 18 ff.)

Designing the Innerferrera Spillway

This article is devoted to the model tests carried out with a view to arriving at the most favourable design of this structure, a minor but very important work of the Valle di lei-Hinter-rhein Development. Sited in a gorge, about 900 m. downstream from the Innerferrera village and 1,000 m. above the Ferrera underground powerhouse, it controls the releases from the Innerferrera balancing reservoir on the Averser-rhein, 300,000 cu. m. in gross, and 230,000 cu. m. in useful capacity. No longer than 54 m. at the crest, for a storage depth of 21 m., its importance is shown by the fact that it must discharge a maximum of 1,100 cu. m. per second without causing any substantial damage to the stream bed. This discharge corresponds to a flow of no less than 70 cu. m. per second per metre width of the sluiceways where most of the energy of the falling water must be dissipated in the stilling pool. The structure has two tainter gates controlling the bottom outlets, each 8 m. wide by 3·5 m. high and exactly above them, two 8 m. wide overflows fitted with automatic flap gates. The bottom outlets and crest overflows are separated by a 4 m. wide pier which stands in the middle of the stream bed. The spillway face is of the Creager type. In the case of simultaneous discharge by bottom outlets and crest overflows, the access of air to the bottom openings would be interrupted, and this could create underpressures or pulsations likely to cause gate oscillations which might result in fatigue fractures. This has been remedied by wedge-shaped baffles fixed on the pier at the end of the spillway face under which air can have unimpaired access to the bottom outlets at flood periods without reducing the overflow discharge. The laboratory tests at the Federal Tech-



Newport News builds 10 unique runners



For Chief Joseph Dam in Washington State

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nical University, Zurich, were carried out on a model of 1 : 50 scale reproducing the storage area, the spillway as well as a 350 m. long stretch of the tail channel. They proved once more that model tests enabled the behaviour of the structure and natural discharge conditions, especially at periods of extreme floods, to be more accurately determined than would be possible by the most careful computation. Particular attention was devoted to the investigation of silting and flushing and the model tests established that (1) all demands made on the spillway regarding discharge capacity and energy dissipation can be met; (2) any lasting silting of the storage space of the reservoir can be avoided with bottom outlets of the size adopted provided that flushing through them is carried out regularly at flood discharges of 40 to 50 cu. m. per second (R. Härri, Motor Columbus A.G., Baden, *Schweizerische Bauzeitung*, Vol. 77, No. 34, August 1959, pp. 543-546, 6 ff.)

Testing a 220-kV Transmission Line Tower

An account is given of the test carried out at the testing station of the Kamani Engineering Corporation Ltd., Kurla Works, on a tower designed for the 220 kV transmission line of the Koyna hydro-electric project. Following a description of the testing plant, the various tests—transverse and vertical loading, longitudinal loading, testing load varying between 150% and 250% of design load, broken wire conditions, and destructive test—are briefly described and the results analysed. (G. R. Hukmani, *Indian Journal of Power and River Valley Development*, Vol. VIII, No. 12, p. 69, 5 pp., 6 ff.)

Truyère Basin Development

With the Sarrans-Brommat, Couesque, Cambeyrac and Lardit power plants now in operation, the Truyère Basin's power production at present is 1,300 million kWh a year for an installed capacity of 400 MW. The new Granval dam, nearing completion some 8 miles downstream from the well-known Garabit Viaduct will add another 125 million kWh of its own to the above production figure and increase the output of the downstream chain of plants by 18 million kWh. This is due to the creation, at the head of the river, of a utilisable reserve of 240 million cu. m. of water power. The dam itself presents a characteristic quite uncommon in multiple-arch structures. A straight crest consisting of a massive slab which carries a 5·5 m. wide roadway while a further feature is that the main arch is set up in the axis of the river and is flanked by two arches on the right bank and three on the left bank, and accommodates a cupola-roofed rotunda-like power house. The six arches rest on seven buttresses spaced 5·0 m. between axes and solidly embedded in the foundation rock by grouting. Five of the arches are two-block structures while the main arch is built in three blocks. Water tightness between adjacent blocks and between blocks and buttresses is ensured by copper strips. The crest, 330 m. in length, lies 76 m. above the river bed. The computations of stresses in the dam itself and especially in the linking of the arches with the crest, were checked by tests on small-scale models. Flood discharge will be effected through two passes whose ski-jump outlet channels rest on the two buttresses built in the axis of the river. Each pass is designed for a discharge capacity of 900 cu. m. per second. An outlet conduit and two 3·6 m. dia. 38 m. long penstocks are led through the main arch which also houses the intake in its axis. The generating equipment consists of two 34 MW, vertical shaft Francis turbines operating at 230 r.p.m. from a head of 75 m. and driving each a 38 MW alternator. Power is generated at 10·5 kV and is stepped up to 220 kV in two transformers 10·5/220 kV—38,000 kVA. Equipment is handled by means of a swivelling crane similar in design to conventional travelling cranes but with one end revolving around a central pillar while the outer end runs on a track laid on a cornice of the wall of the circular building. No more than 170,000 cu. m. of concrete entered into the construction of the Grandval dam. This figure illustrates the economy achieved by the advance of dam-building techniques. Thirty years ago, no solution other than a gravity dam would have been adopted on this same site and this gravity dam would have required the placing of at least 450,000 cu. m. of concrete. (*La Technique de l'Eau*, Vol. XIII, No. 148, April 15, 1959, pp. 45-47, 3 ff.)

NOTE: An article on the same subject appears in the undermentioned journal. Besides the unusual arrangement of a circular powerhouse inside one of the arch bays, the article stresses the notable feature of the arch crests, which instead of being horizontal, slope downstream at the same angle as the slope of the buttresses, thus leaning over the buttresses between adjacent arches. Openings are plugged with the gravity section walls whose weight improve the overall stability of the dam. (*Engineering News-Record*, Vol. 163, No. 6, August 6, 1959, pp. 49-50, 2 ff.)

Harnessing the Lasele Falls

This handsome publication in English of the Swedish State Power Board is devoted to the Lasele Plant, situated on the Fjälsjö River some five miles upstream of its confluence with the Ångerman River. The latter pours into the Gulf of Bothnia about halfway between Stockholm and Haparanda. The power pool is created by a 290 m. long, 25 m. high rockfill dam on the left bank with an adjacent three-bay spillway block. The bay nearest to the right bank is 15 m. wide by 5 m. high and is intended as an emergency timber pass, while the other two, also 15 m. wide but 8·3 m. high, act as normal spillways. At normal flood periods, timber is passed through a channel whose inlet is sited close to the emergency pass. The power intake proper is close to the right bank of the river whose boulder ridge is protected from seepage by an impervious blanket extending 150 m. upstream of the dam. The two 7·6 m. diameter penstocks, blasted through rock, have a steel-plate lining varying in thickness between 12 mm. at the top, and 25 mm. at the downstream ends. The underground powerhouse follows the general pattern adopted by the Swedish State Power Board so that the machine hall with its adjacent control room and the transformer room are located downstream and roughly at 90° angle to each other and all have a natural rock roof. There are two 70 MVA generating sets driven by Kaplan turbines operating at 150 r.p.m. from a net head of 52 m. so far a Swedish record for Kaplan heads. The draft tubes discharge into a 150 m. long surge basin whose cross-section narrows from 318 sq. m. upstream to

215 sq. m. downstream and which connects with the 1,560 m. long tail-race tunnel. This tunnel discharges into a 1,100 m. long canal which returns the water from the turbines to the river. The detailed description of the installations is preceded by paragraphs on the hydrological and geological characteristics of the development and is followed by a section dealing with the measures taken for the protection of the landscape. (Sten Elfman, Swedish State Power Board, Stockholm, *Blue-White* Series, No. 21, 16 pp., 21 ff.)

The Grande Dixence Dam

This paper sets out to show some of the aspects of recent Swiss hydro-electric development with particular reference to the construction of the Grande Dixence Dam. The dam will create a storage reservoir of 322,000 acre/ft. at 7,700 ft. above sea level to supply two powerhouses aggregating 840,000 kVA, from a total head of about 6,000 ft. In height, its 930 ft. above lowest foundation level will leave Hoover Dam, until now the highest dam in the world, 200 ft. behind. Following a short historic review of the Grande Dixence Development the author deals briefly with the economics of the project and with the difficulties encountered in the supply of materials and labour on such a high mountain site. Stress is laid on the construction itself. With no diversion works required, no powerhouse immediately downstream and with the intake, spillway and sluiceway as relatively minor works, construction has been essentially a straightforward one, in spite of the fact that two seasons' work was necessary to remove 650,000 cu. yd. of overburden at the site, and 250,000 cu. yd. of rock from the steep abutments. Concrete placing is described in detail and shows the dam as a gravity structure broken into large rectangular pouring blocks with transverse joints every 52 ft. and longitudinal joints at a maximum spacing of 130 ft. One of these blocks is with a lift of 10·5 ft., indicating that a high lift is almost universally adopted in high altitude Swiss dams and takes 2,600 cu. yd. The height of lift is dictated only by the spacing of cooling coils. If possible, concrete temperature is kept to about 4° C., i.e., about the average snowmelt temperature, but rises up to 25° C. immediately upon placing only to decrease then to about 22° C. when the next lift is poured three days later. The high lifts require strong falsework which is generally expensive but, in this high dam, proved economic. By using a stepped profile of block construction instead of the conventional castellated profile, substantial economy has been achieved. In this method the upstream and downstream faces of the blocks are boxed with conventional cantilevered shutters, one transverse face being formed by the step above, and the other boxed with a movable form sitting on the adjacent block. Production of concrete is described in relation to the winning and preparation of the aggregate. Nearly all concrete is low-slump mass concrete. At the outset, two two-man (260 lb.) hand vibrators were used for each 8 cu. yd. bucket of concrete poured. Now, following the Mauvoisin example, only inaccessible corners are hand-vibrated; otherwise, vibrators mounted on the bladeframe of a small tractor are the rule. Another small bulldozer is used to spread the concrete. In the 1957 season of six months, 1,300,000 cu. yd. of concrete was placed, with a maximum of nearly 12,000 cu. yd. for one day. In spite of bad weather in 1958 the volume placed reached 1,130,000 cu. yd. Originally planned to be built in stages with intermediate crest levels, construction is now proceeding without a break. With the first 650 ft. of height reached the dam is now in service and final completion is expected in late 1961. (C.P.O. Turner, Ministry of Works, Wellington, *N.Z. Engineering*, April 15, 1959, p. 111, 5 pp., 7 ff.)

Jaldhaka Project

This hydro-electric project is the first major venture in West Bengal to harness a sizeable potential and open up the way to industrialisation in a comparatively backward region. The reach of the river between Bindu and Naksal is proposed to be developed in three stages. The first will utilise a drop of 580 ft. between Bindu Khola confluence and Biru Khola confluence. The second stage consists of a further drop of 300 ft. up to Naksal confluence, while the third stage envisages to utilise a drop of about 1,300 ft. on the Nichu, the tributary of Jaldhaka river which forms the northern boundary of West Bengal with Bhutan. The power available at 50% load factor in the above three stages is 36 MW, 24 MW and 24 MW respectively, inclusive of 18 MW and 12 MW of secondary power in the first and second stages. The main works contemplated in the first stage are a short diversion weir across the Jaldhaka river below its confluence with Bindu Khola which will divert water into a composite supply system consisting of a reinforced cement concrete duct, a short open flume, and a two-and-half mile tunnel leading to a surge chamber from which four steel-lined tunnels will connect each with a steel penstock. The powerhouse will accommodate four turbo-alternator sets, 9 MW each, and the adjoining outdoor switchyard will step up the 11,000 V generated power to 66,000 V. (Monoranjan Datta, *Indian Journal of Power and River Valley Development*, Vol. IX, No. 1, January 1959, p. 1, 9 pp., 5 ff.)

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Announcements for this column can be accepted up to the 8th of the month for the following month's issue. The charges are fourpence per word with a minimum of 10s. Box No. facilities 2s. 6d. extra. In order to avoid accountancy it would be appreciated if instructions to insert were accompanied with remittance.

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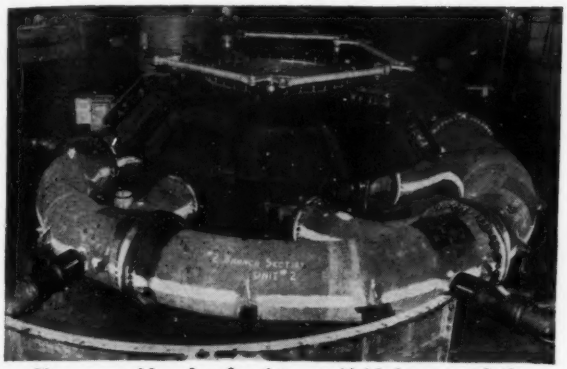
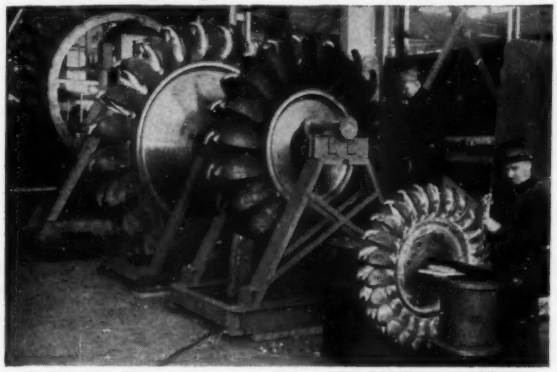
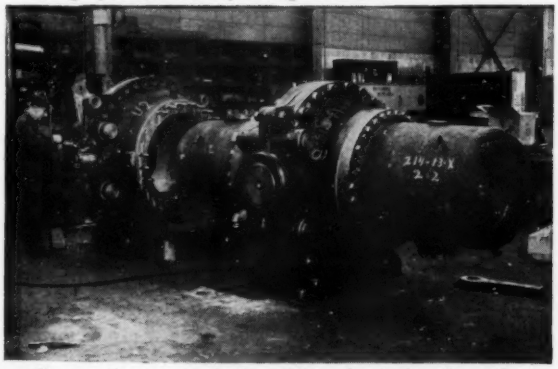


Shop assembly of a five-jet manifold for one of the two 28,750 h.p. Impulse Turbines for Kundah No. 1.



Polishing the runners—at left, a 50,000 h.p. turbine, at right, one of the 28,750 h.p. runners.



Shop-testing one of the 2'6" spherical valves for Kundah No. 2 at a pressure of 2,140 psi.

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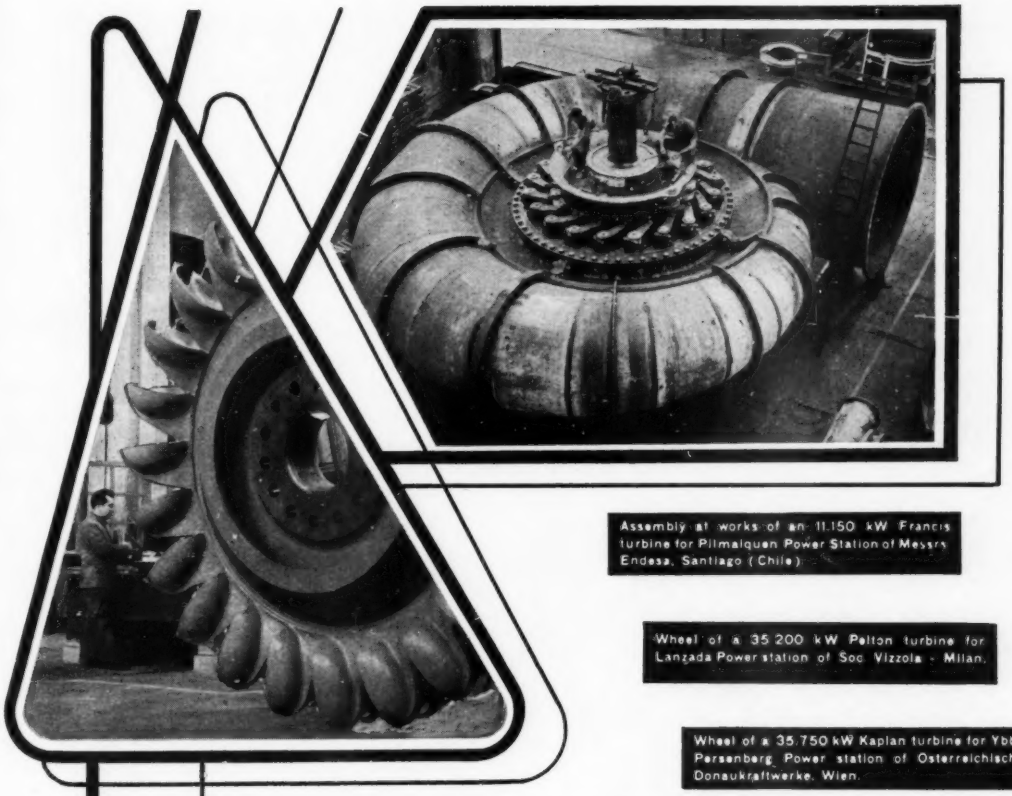
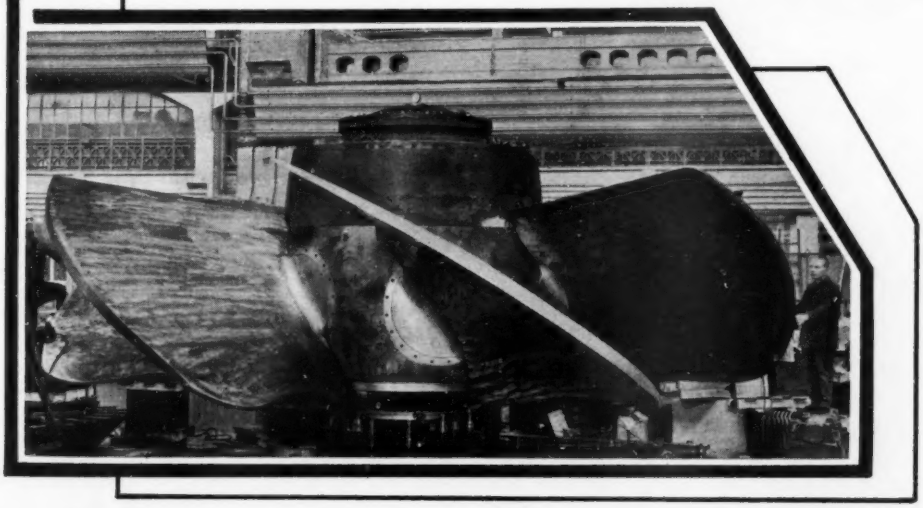


Assembly at works of an 11.150 kW Francis turbine for Pilmaiquen Power Station of Messrs Endesa, Santiago (Chile).

Wheel of a 35.200 kW Pelton turbine for Lanzada Power station of Soc. Vizzola - Milan.

Wheel of a 35.750 kW Kaplan turbine for Ybbs Persenberg Power station of Osterreichische Donaukraftwerke, Wien.



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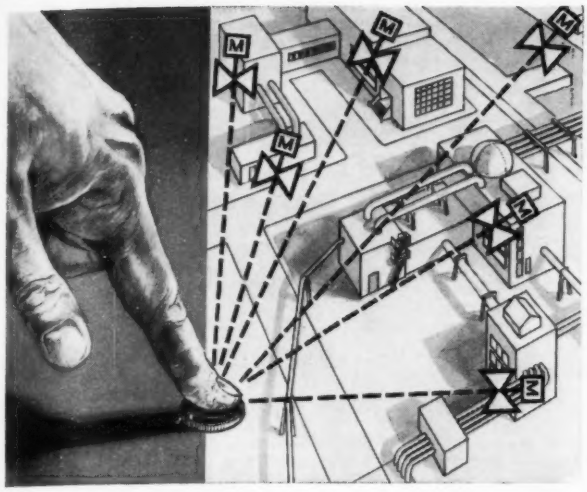

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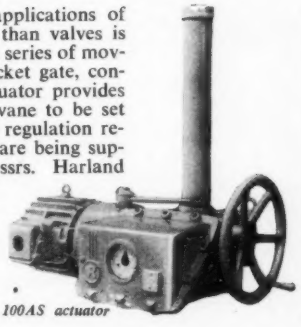

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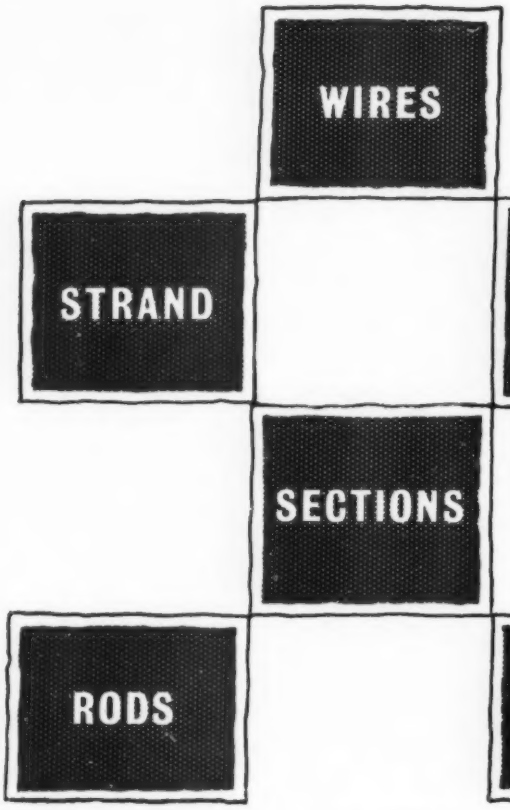

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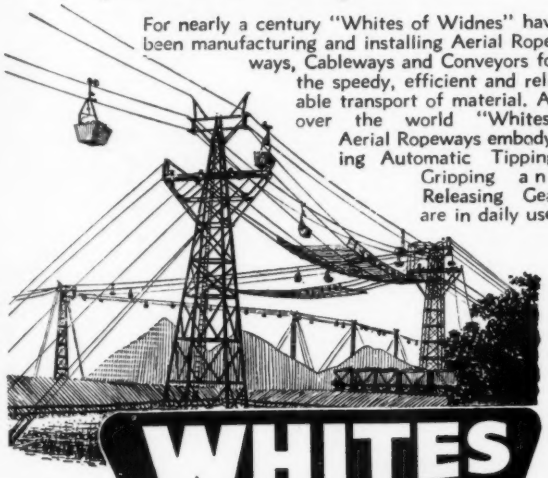

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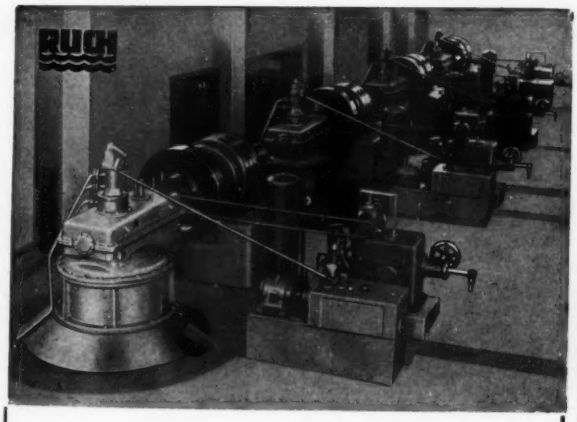

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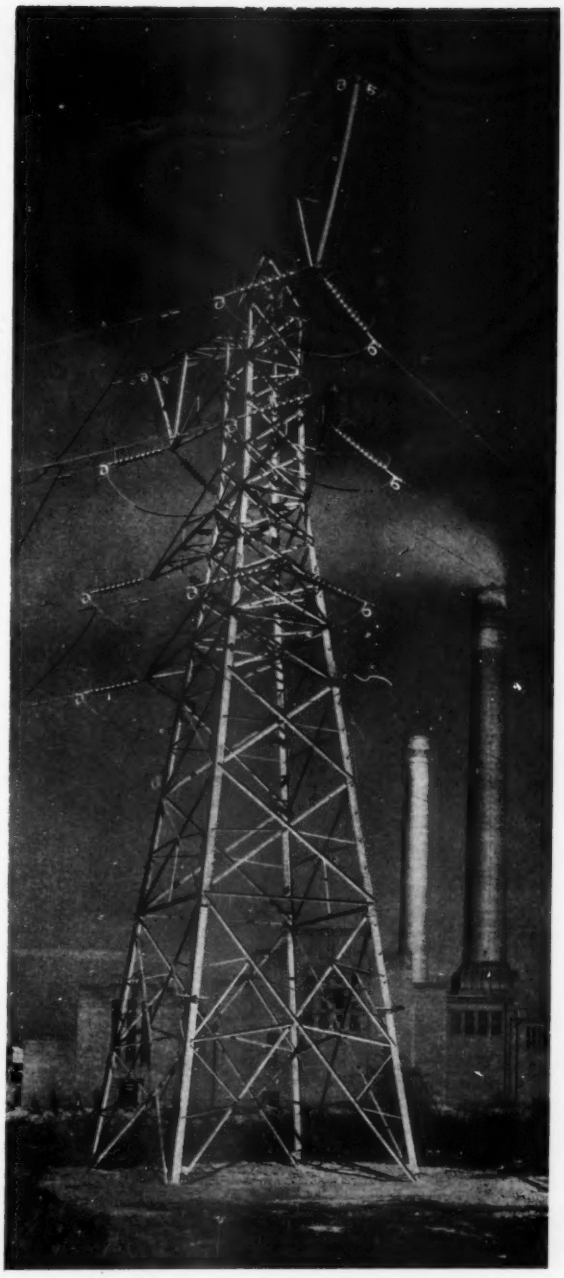

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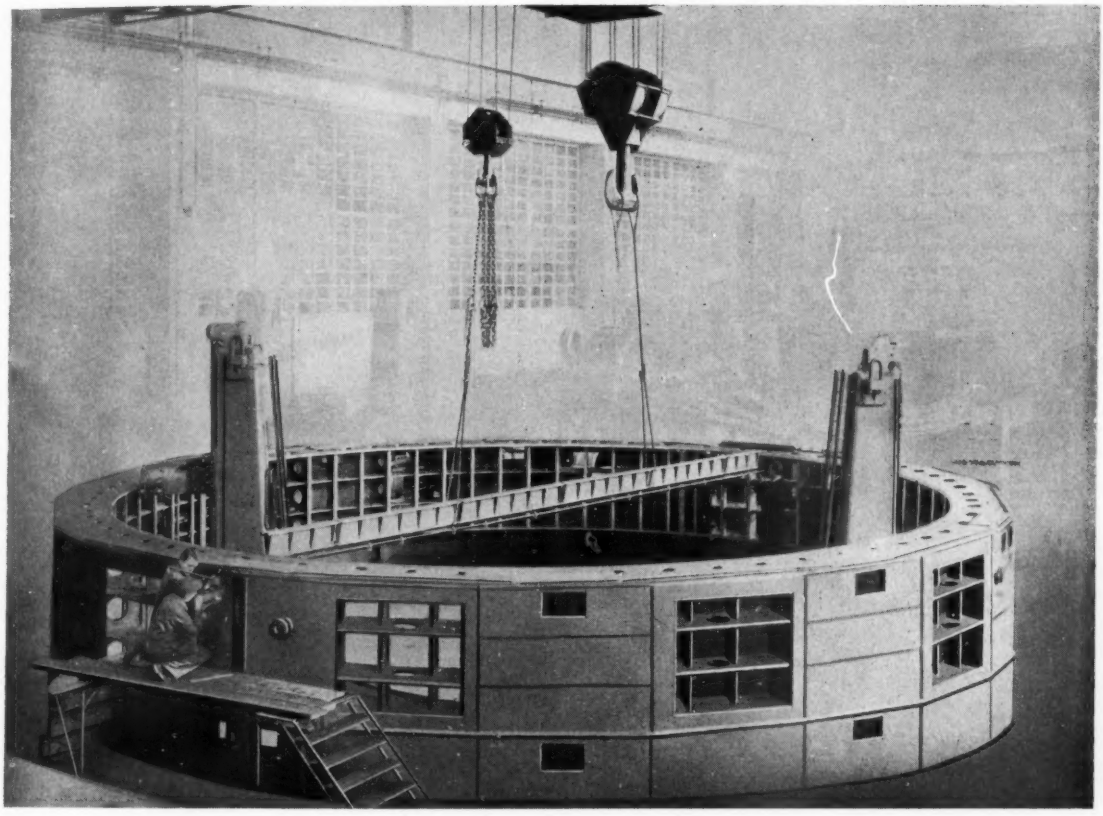
LOW-SPEED WATER-WHEEL-DRIVEN GENERATORS



Measuring the bore on the casing of a 50-MVA umbrella-type generator, 75 rpm, for Järkvissle Power Station of the Swedish State Power Board. Bore: 12 m (39 ft.). Outside diameter: 14 m (46 ft.)

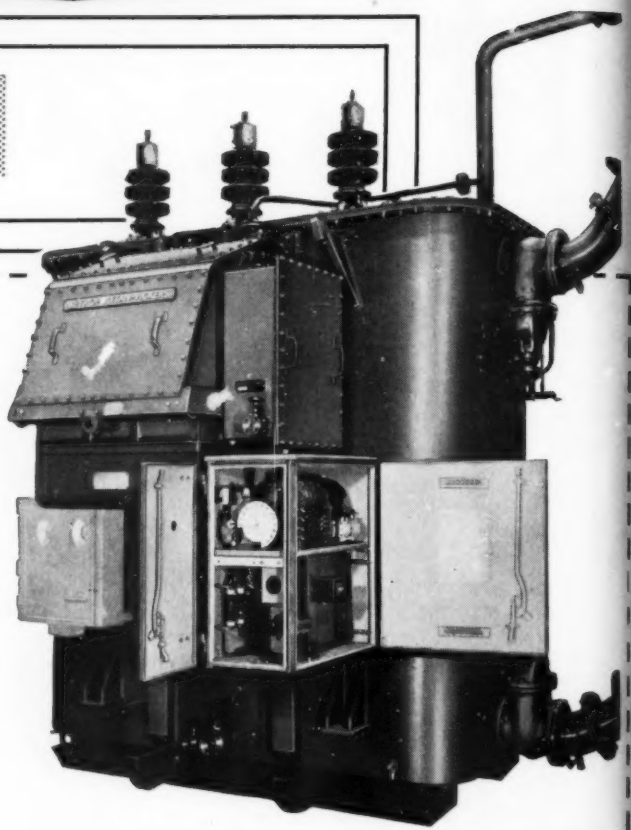
ELIN-UNION

Aktiengesellschaft für elektrische Industrie

Vienna-Austria

On-load Tap Changers

TYPE M



Designed for use on transformers of up to 275 kV, the Type M tap changer has a simple and reliable operating mechanism which ensures completion of tap change. Transition resistors—liberally rated and mechanically robust—minimize arcing and burning at the contacts of the diverter switches, and ease the switching duty. The complete tap changer with its motor driving gear is compactly assembled as a single, iron-clad unit and bolted to a flange on the transformer tank.

Other important features:

- *Any number of steps up to 18 can be accommodated*
- *Once initiated, a tap change cannot be interrupted*
- *Continuity of supply maintained during a tap change*
- *Separate drop-down tank allows convenient inspection of diverter switches*
- *Diverter switches are rapid in operation*
- *Contact bounce at diverter switches eliminated*
- *Minimum voltage variation during a tap change*
- *Moving contacts have single bolt fixing and are accessible from the front.*

Associated Electrical Industries Limited
TRANSFORMER DIVISION
Manchester 23 and Rugby

E/A580